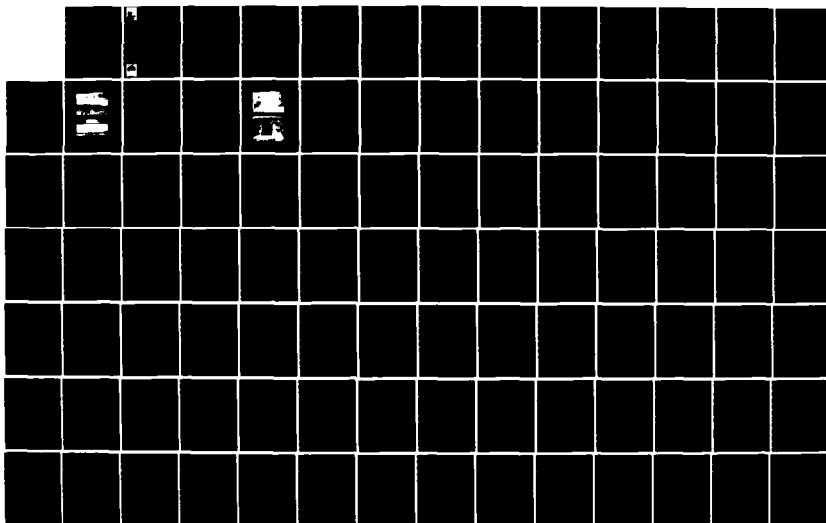SYNTHESIS OF RAILROAD DESIGN METHODS TRACK RESPONSE
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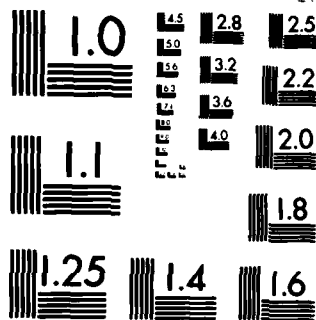UNCLASSIFIED   MAR 85 WES/MP/GL-85-3   F/G 13/13   NL

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SYNTHESIS OF RAILROAD DESIGN METHODS, TRACK RESPONSE MODELS, AND EVALUATION METHODS FOR MILITARY RAILROADS

by

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Final Report

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report presents the results of a state-of-the-art review performed in the areas of railroad track structures, railroad design, and railroad evaluation. Described and discussed are the components comprising the railroad track system, railroad design procedures, analytical track response models, track performance models, methods of structural evaluation, rail defect testing, methods of functional evaluation, and the effect of heavy axle loads.

(Continued)

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20. ABSTRACT (Continued).

Based on the results of this study an analytical track response model was chosen for use in future research into the evaluation of track structures. Three methods for testing track structures were chosen for additional investigation. An evaluation program comprising both structural (load-carrying capacity) and functional evaluation is proposed.

PREFACE

The research effort reported herein was conducted and this report was prepared for the Office of the Chief of Engineers (OCE), under RDT&E Project No. 4A162719AT40 entitled "Improved Practice for Military Railroad Rehabilitation and Maintenance for Army Mobilization." The Technical Monitor at OCE was Mr. R. W. Williams.

The literature review reported herein was performed during the period November 1983 to September 1984 by Mr. D. M. Coleman, Pavement Systems Division (PSD), Geotechnical Laboratory (GL), US Army Engineer Waterways Experiment Station (WES). Other personnel of the PSD, WES, who assisted in the study that lead to the preparation of this report were Messrs. J. W. Hall, Jr., R. D. Jackson, and S. D. Kohn. This report was prepared by Mr. Coleman under the direct supervision of Mr. R. W. Grau, Chief, Prototype Testing and Evaluation Unit, PSD, and the general supervision of Mr. H. H. Ulery, Chief, PSD, Dr. T. D. White, former Chief, PSD, and Dr. W. F. Marcuson III, Chief, GL.

The Commanders and Directors of WES during the conduct of this study and preparation of this report were COL Tilford C. Creel, CE, and COL Robert C. Lee, CE. Mr. F. R. Brown was Technical Director.

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CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric)
units as follows:

Multiply	By	To Obtain
cubic inches	16.38706	cubic centimetres
feet	0.3048	metres
inches	2.54	centimetres
kips (force)	4.448222	kilonewtons
kips (force) per inch	175.1268	kilonewtons per metre
miles (US statute)	1.609347	kilometres
mils	0.0254	millimetres
pounds (force)	4.448222	newtons
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6894.757	pascals
pounds (force) per yard	0.5932764	kilograms per metre
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic inch	27.6799	grams per cubic centimetre
square inches	6.4516	square centimetres
tons (2,000 lb, mass)	907.1847	kilograms

SYNTHESIS OF RAILROAD DESIGN METHODS, TRACK RESPONSE MODELS, AND EVALUATION METHODS FOR MILITARY RAILROADS

PART I: INTRODUCTION

Background

1. A large part of the military railroad trackage currently in exist-
ence throughout the continental United States (CONUS) was constructed to meet
the logistical demands of World War II (WW II). Much of this track was built
with an anticipated usage of 5 years. Due to the time limitations, manpower
shortages, and material shortages existing at that time, much of the material
used in constructing the military trackage was surplus material that had been
previously used by commercial railroads. Rail sizes used in the original
construction of military trackage ranged from 50 to 105 lb/yd* with the pre-
dominant sizes being 70, 75, 80, and 85 lb/yd. Since WW II these railroads
have, for the most part, supported only light loads and light to medium
traffic densities. As a result of this limited utilization, the military rail
system has received limited visibility and subsequently limited attention
during the intervening years. The overall result of the limited use of the
military railroads is that many installations often lack funding and/or
trained personnel for railroad maintenance; therefore only minimal maintenance
is performed. To further compound the problem, the gross weight of railroad
cars using this track has increased from an estimated average of 50 to 60 tons
during WW II up to 100 to 140 tons today. Some cars in the current military
inventory exceed 200 tons gross weight. Current military preparedness mis-
sions require the use of the military railroad system; however, limited utili-
zation, limited funding for maintenance and repair, and lack of trained per-
sonnel have often resulted in an installation's railroad being in some unde-
fined state of repair which may or may not support the loads and traffic im-
posed upon it while carrying out the installation's mission.

2. It is with this background that in recent years action has been

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* A table of factors for converting non-SI units of measurement to SI
(metric) units is presented on page 4.

5

taken to upgrade the military's railroad trackage to prepare for future mission requirements. The development and implementation of the US Army Railroad Maintenance Standards is one action which will have an immediate positive effect on the military's railroad trackage. These standards identify the maintenance levels that must be maintained to ensure safe and efficient completion of an installation's mission. The US Army Engineer Waterways Experiment Station (WES) is monitoring the field testing of geotechnical fabrics at the Association of American Railroads Transportation Test Center to determine what fabric type, size, and placement depth will result in the best utilization of these materials under military railroad track. The most recent testing program is a laboratory study to determine the remaining fatigue life of the lightweight rail (rail sizes 90 lb/yd and below) that exists on approximately 80 percent of the military's trackage. This study on lightweight rail is being conducted by the US Army Construction Engineering Research Laboratory (CERL) and the US Army Engineer Division, Huntsville, to determine if the lightweight rail that is presently in use is structurally adequate to support anticipated mission traffic.

Purpose

3. The purpose of this investigation was to determine, by means of a literature review, the state of the art in the area of railroad design and evaluation. Specific objectives of this study were to (a) review railroad design procedures, (b) review various models for predicting track response, (c) review existing procedures for evaluation of track, (d) determine requirements for predicting railroad track performance, (e) determine requirements for development of a comprehensive railroad evaluation methodology, and (f) identify areas where additional research is needed.

Scope

4. In the preparation of this report 153 reports, papers, and articles were reviewed covering a variety of subjects including railroad design, loads and stresses in railroad track structure, ballast and subgrade response, track response models, structural (load-carrying capacity) testing, track geometry inspection, rail defect testing, heavy wheel/axle loads, maintenance needs

6

determination and planning, and general railroad research. Telephone calls and personal visits with personnel from various commercial railroads, colleges and universities, state departments of transportation, the Federal Railroad Administration (FRA), the Association of American Railroads (AAR), the American Railway Engineering Association (AREA), and various private firms involved in railroad testing provided much valuable information for this report. Appendix A presents a complete list of texts, reports, papers, and articles reviewed during this literature review. Appendix B presents a list of commercial railroads, colleges and universities, government agencies, railroad organizations, and private firms contacted during this literature review.

5. This report summarizes the results of this literature review. Described and discussed are:

 a. The components comprising the railroad track system.
 b. Railroad design procedures.
 c. Analytical track response models.
 d. Track performance models.
 e. Methods of structural evaluation.
 f. Methods of rail defect testing and geometry testing.
 g. The effect of heavy axle loads.

Recommendations for development of a comprehensive track evaluation program are presented along with areas for additional research.

PART II: COMPONENTS OF THE RAILROAD TRACK SYSTEM

Overview

6. The conventional railroad track system is a structure constructed to (a) provide guidance for locomotive and rolling stock wheels, (b) support the loads resulting from these wheels, and (c) distribute the wheel/axle loads throughout the track structure in such a manner as not to overstress the subgrade material. The conventional railroad track structure is composed of rails, tie plates, ties, ballast, subballast, and the subgrade. The subballast layer is not present in all track structures, and use of the term "ballast" in this report includes the subballast where present. Figure 1 shows the typical railroad track structure.

7. The distribution of load from the wheel to the subgrade is one of the most important functions of the track structure. A 30,000-lb wheel load acting on an approximately 0.5-in.^{2} contact area creates a stress on the rail of 60,000 psi. Most reasonably firm subgrades will support vertical stresses in the range of 10 to 20 psi. The track structure must reduce the wheel load stress on the rail to a stress that the subgrade will support. If the subgrade will not support the loads produced by the track structure and train, loss of surface, line, and gage, pumping, ballast fouling, and roadbed subsidence will occur. Each of the primary components of the track structure will be discussed in detail in the following paragraphs.

Rails

8. Railroad rail is steel rolled into what is normally referred to as a "tee" configuration and usually is produced in 39-ft lengths. Rail is rolled into different dimensions and shapes which are referred to as weight and section, respectively. The weight of a rail is based on how much a rail weighs in pounds per yard of length. The rail section is the pattern design or cross-sectional dimension of the rail. Each rail carries a brand rolled into its web as raised letters and figures, which identifies the rail weight per yard, rail section number, type rail, manufacturer name and mill, and year and month rolled. On the web opposite the brand is the stamp which identifies the serial heat number, ingot number as cast or rolled, and the position of the

8
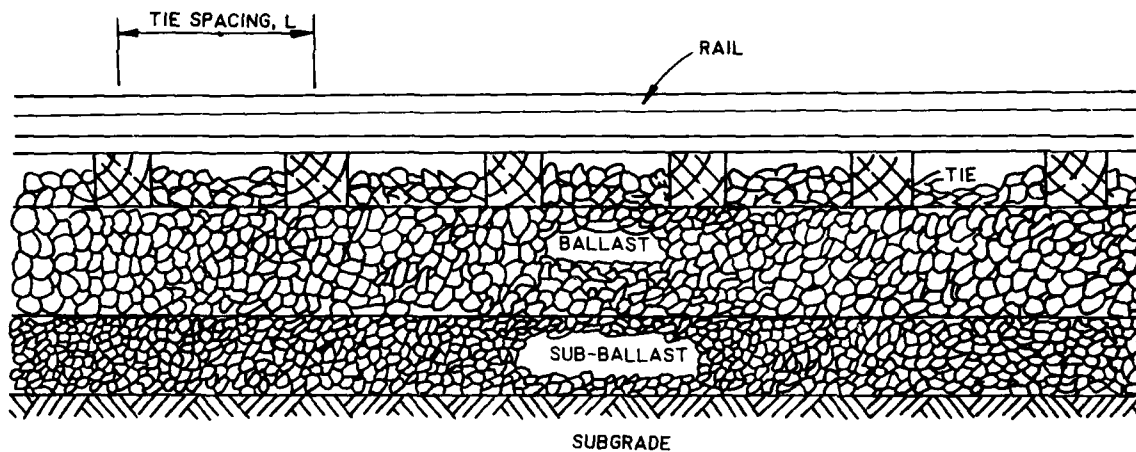
(a) LONGITUDINAL SECTION
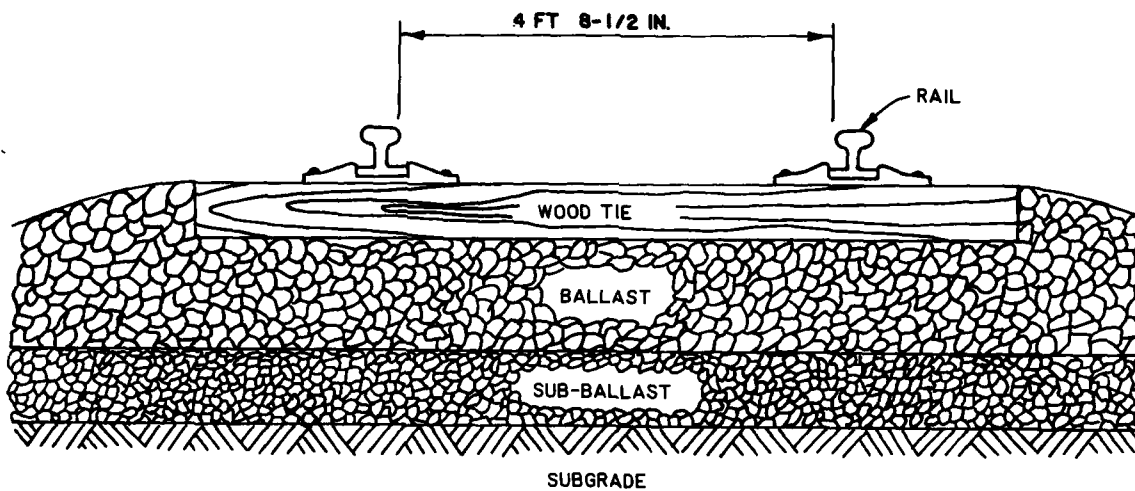
(b) TRANSVERSE SECTION

Figure 1. Typical railroad track structure

rail with reference to the top of the ingot. Figure 2 shows a typical brand, while Figure 3 shows a typical stamp. Excellent references on the composition, manufacture, and identification of railroad rail are found in the "Sperry Rail

Figure 2. Typical brand

Figure 3. Typical stamp

10

Defect Manual" (Sperry Rail Service 1964), "Railroad Engineering" (Hay 1982), and "Rail Track Fundamentals" (US Department of Transportation 1977).

9. Relayer rail is a secondhand rail that is in reusable condition as is or after cropping off the ends. Reclaimed relayer rail in good condition is often used to replace damaged or defective rails, or for spot rail renewals.

Industry standards

10. Commercial railroads use rail weights ranging from 90 to 155 lb, with the most common rail weights in use being the 115RE, 119RE, 132RE, 136RE, and 140RE sections (US Department of Transportation 1977). In "Technical Data Bases Report: Ballast and Foundation Materials Research Program" (Robnett et al. 1975), the range of rail weights currently in use is given as 90 to 155 lb, with lighter sections in use on some low-density branch lines. This report lists typical rail weights currently in use by commercial railroads as 115 or 119 lb for light- and medium-density lines, 132 or 136 lb for high-density lines, and 140 lb or heavier for lines with very high traffic densities. The AREA has recommended that the 90RA-A section be the smallest rail section used. Rail sections recommended for use by the AREA are 140RE, 136RE, 132RE, 119RE, 115RE, 106CF&I, 100RE, and 90RA-A. These recommendations are based on (a) future availability of these sections for small orders, (b) ready availability of other track materials, special trackwork, and accessories for these sections, and (c) supplier experience in producing these sections (American Railway Engineering Association 1984). Many commercial railroads use continuous welded rail (CWR) on lines with high traffic densities and speeds. CWR is produced by welding standard rail lengths into a continuous strand up to 1,440 ft long. CWR has the advantages of providing longer rail service life due to elimination of joint wear and end batter; easier, quieter riding; reduced rolling stock wear; and reduced track maintenance. Disadvantages include difficulty in laying, difficulty in repairing broken rail, and the precautions that must be taken to prevent excessive compression or expansion due to thermal effects.

Army standards and usage

11. In accordance with Army Regulation 420-72 (Headquarters, Department of the Army 1976, the standard rail for Army use is the 90-lb ARA-A section; however, heavier rail sections may be used to meet the minimum requirements of a serving commercial railroad or when approved by the operating agency

(Headquarters, Department of the Army March 1976). "Railroad Design and Con-
struction at Army and Air Force Installations" (Headquarters, Departments of
the Army and the Air Force July 1980) specifies that for new construction new
or relayer 90 ARA-A or 115 AREA rail sections should be used; however, new
rail is preferred. The 115-lb section is specified for use on main lines,
access tracks where traffic is heavy, or where the design train speed is
greater than 40 mph. The 90-lb section is specified for all tracks that do not
justify the use of 115-lb rail. Technical Manual TM 5-627/MO-103/AFM 91-33,
"Maintenance of Trackage," (Headquarters, Departments of the Army, Navy, and
Air Force, January 1980) specifies the 90-lb ARA-A section for replacement of
worn or substandard trackage, with heavier rail allowable if conditions war-
rant and the appropriate headquarters approves.

12. Inspections of tracks at various Army installations indicate a wide
range of rail sections currently in use. These sections range from 55 to
136 lb with the majority being 80-, 85-, and 90-lb rail sections. Much of
this lightweight rail was relayer rail at the time the tracks were constructed.
In addition, much of the lightweight rail in service was rolled prior to 1936,
about which time controlled cooling of railroad rail became standard steel in-
dustry practice. These rails that were not control cooled during manufacture
have a greater risk of possessing internal mill defects that can result in
transverse fissures and rail breakage.

Tie Plates

13. The tie plate is a steel plate interposed between the bottom of the
rail and the top of a wooden tie whose primary purpose is to protect the tie
from mechanical wear while establishing and maintaining the integrity of the
track. The tie plate is larger than the rail base to distribute the rail
loads over a greater tie area at a lower unit pressure. Tie plates are com-
monly rectangular in shape and vary in width from 6 to 9-1/2 in. The tie
plate length is designed to afford the required bearing area and to provide a
safe margin of steel between the plate edges and the holes punched for the
track spikes. Tie plates in general use range from 9 to 14 in. long and vary
from 9/16 to 1 in. thick. Tie plates may be either single shouldered (Figure 4)
or doubled shouldered (Figure 5). The purpose of these shoulders is to hold
correct line and gage. Most tie plates are canted with a 1:30 or 1:40 cant

12

Figure 4.   Single shouldered tie plate

Figure 5.   Double shouldered tie plate

13

to account for the resultant of vertical and lateral wheel load forces. The 1:40 cant tie plate is recommended by the AREA and is in almost universal use on North American Commercial Railroads. However, 1:30 cant tie plates are also in use with good results (Hay 1982).

Industry standards

14. Most commercial railroads use tie plates on their main-line wooden tie tracks. The tie plates typically used are 7-3/4 to 8 in. wide and 12 to 14 in. long. Tie plates 18 in. long are used on a few high-degree curves. The exception to this is low-density branch lines and yard tracks where tie plates are sometimes not used.

Army standards and usage

15. Army Regulation 420-72 (Headquarters, Department of the Army March 1976) outlines the conditions under which tie plates will be used as: "(a) for treated ties being installed in trackage and (b) for treated ties in places where experience shows a need." The regulation also states that consideration will first be given to running or access and classification yard trackage. Technical Manual 5-850-2/AFM 88-7, Chapter 2 (Headquarters, Department of the Army and the Air Force July 1980) specifies that with new 90-lb ARA-A rail, new tie plates corresponding to AREA Plan No. 1, with "A" punching be used. With new 115-lb AREA rail, new tie plates corresponding to AREA Plan No. 4 with "A" punching should be used. With relayer rail, previously used tie plates may be used if they are of the proper size and punching; however, these plates should not be smaller than 7-1/2 by 10 in. for 85-lb rail or 7-1/2 by 11 in. for 110-lb rail. A majority of the army trackage has tie plates corresponding to the rail size being used; however, some low density or storage tracks do not have tie plates.

Crossties

16. Crossties or ties are the structural members that support the rails. The purpose of ties is to (a) hold proper gage and line, (b) transmit axle loads from the rail to the ballast with reduced unit pressures, and (c) reduce lateral, longitudinal, and vertical movement of the track. Ties are typically made of wood although steel ties and concrete ties are in use. Steel ties have the advantage of long life which reduces the labor/maintenance cost; however, they have never been used extensively in the United States

14

because of higher initial costs and difficulty in removal and replacement should they become bent. Concrete ties are in general use throughout Europe and are used by some railroads at select locations in the United States and Canada. Some of the advantages usually given for concrete ties are:

    a.   Their larger effective bearing area usually permits wider tie spacing, reducing the number of ties required per mile.

    b.   Their increased weight contributes to greater lateral stability.

    c.   They allow the use of lower maintenance, adjustable rail fasteners instead of cut spikes.

    d.   Concrete is resistant to chemicals, weather, abrasion, and insects providing potential for long service life.

The principal disadvantages of concrete ties are:

    a.   The increased weight makes them more difficult to handle and install, particularly for spot renewal.

    b.   Attachment of rail fasteners to the tie is a serious design problem.

    c.   The rigid structure of concrete ties makes them more susceptible to major damage during handling or a derailment.

17. The large majority of ties in use in the Unites States today are wooden. Wooden ties provide elasticity to the track structure, are relatively inexpensive, are easily worked, and have a relatively long life. The most commonly used woods in tie production are Douglas fir, red oak, white oak, and longleaf pine, although several other varieties of wood are used. Tie sizes vary in cross section from 6 in. thick by 6 in. wide to 7 in. thick by 9 in. wide. Tie lengths vary from 8 to 9 ft, although switch ties (ties placed in turnouts, crossovers, and crossings) vary from 8 to 22 ft and bridge ties are usually 10 in. wide by 10 in. deep and 10 to 12 ft long. Wooden ties are treated with a preservative, usually creosote, to protect them from destruction by insects and decay. Table 1 presents the tie dimensions recommended by the "Manual for Railway Engineering" (American Railway Engineering Association 1984). Sizes 1, 2, and 3 are usually used on siding, while sizes 4 and 5 are used for main-line track.

18. Because ties help distribute axle loads from the rail to the ballast section, the more ties per section of rail the lower the load on the ballast section at any given point. The recommended maximum and minimum number of ties per 39-ft rail length is 24 and 20, respectively. This number of ties per rail results in a center-to-center tie spacing of 19-1/2 to 24 in. "Rail Track Fundamentals" (US Department of Transportation 1977) provides an

excellent discussion on tie defects, defining failed ties, and tie renewal considerations.

Industry standards

19. Wood ties are used almost universally on the commercial railroads of the United States. Most railroads use AREA sizes 4 and 5 on main-line tracks with lengths of 8, 8-1/2, or 9 ft. The AREA recommends 9-ft ties for all new construction and rehabilitation; however 8-1/2-ft-long ties are commonly found on high-density lines. Center-to-center tie spacings of 19-1/2 and 22 in. are the spacings most frequently used on commercial main-line track. This spacing results in 24 and 22 ties, respectively, per 39-ft rail (Robnett et al. 1975). The minimum safety standard prescribed by the FRA in the "Track Safety Standards" (US Department of Transportation 1982) requires five nondefective ties per 39-ft segment of Class 1 track,* with 8 nondefective ties per 39 ft of Classes 2 and 3 track.

20. Concrete crossties have been installed in test sections by various railroads including the Alaska Railroad, Santa Fe, Chessie, and the Norfolk and Western Railroads. A few railroads including the Florida East Coast and the Kansas City Southern have large amounts of concrete tie track. Center-to-center tie spacing with concrete ties is in the 24- to 27-in. range with closer spacing on curved track (Hay 1982).

Army standards and usage

21. Army Regulation 420-72 (Headquarters, Department of the Army March 1976) specifies that wooden ties treated with an "acceptable preservative" will be used for replacement of deteriorated ties in existing track and for track extensions. Installation of used ties is to be confined to light traffic lines, sidings, and dead storage tracks. Concrete ties may be used in critical maintenance areas if the longer life of ties is economically justified. For main lines, access tracks, heavy traffic tracks, and tracks where the design train speed is greater than 40 mph, TM 5-850-2/AFM 88-7, Chapter 2 (Headquarters, Departments of the Army and the Air Force July 1980) requires the use of wooden ties at least 7 in. thick by 8 in. wide by 8-1/2 ft long spaced 22 to 24 ties per 39-ft rail (20 to 19-1/2 in. center-to-center). For low

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* If track meets all the safety requirements, the maximum allowable speed for freight trains is 10 mph for Class 1 track, 25 mph for Class 2 track, 40 mph for Class 3 track, 60 mph for Class 4 track, 80 mph for Class 5 track, and 110 mph for Class 6 track (US Department of Transportation 1982).

use trackage wooden ties 6 in. thick by 7 or 8 in. wide by 8 ft long spaced 20 to 22 ties per 39-ft rail (24 to 21 in. center-to-center) should be used. US Army Corps of Engineers Guide Specification, "Railroads," CEGS-02850 (Headquarters, Department of the Army May 1982) requires that wooden ties conform to Chapter 3, Part 1 of Manual for Railway Engineering (American Railway Engineering Association 1982). The "US Army Rail Maintenance Standards" (Headquarters, Department of the Army in press) outlines the minimum standard for ties as: "No more than 3 consecutive defective crossties...in tangent track having 90-lb rail or heavier." For tangent track having rail less than 90 lb, for curved track, or in turnouts the maximum number of consecutive defective ties is two. The maximum center-to-center spacing of ties should not exceed 22 in. For rail less than 90 lb the maximum center-to-center spacing should not exceed 19-1/2 in. The center line of at least one nondefective tie should be within 18 in. of a rail joint for track with 90-lb or greater rail. For track with rails less than 90 lb, all ties within 18 in. of either side of a rail joint must be nondefective.

22. With the exception of one installation that has placed concrete ties under several road crossings, all of the ties on Army trackage are wooden. The overall condition of ties on Army trackage ranges from very poor to excellent, depending on the installation, track usage, maintenance funds, and other factors.

Ballast and Subballast

23. Ballast is a selected material placed on the roadbed for the purpose of holding the rail-tie system in line and surface. Subballast is any material superior to the subgrade material which is placed on the subgrade of the roadbed and below the top ballast to provide for better drainage, prevent upheaval by frost, and better distribute the load over the roadbed. The functions of the ballast layer in the track structure are to (a) transmit load uniformly to the subgrade, (b) restrain track from lateral, longitudinal, and vertical movement, (c) provide adequate drainage, (d) provide uniform support for ties to provide proper surface and line, (e) minimize climatic influences, and (f) facilitate maintenance operations. Characteristics of a good ballast material are strength, toughness, durability, stability, drainability, cleanability, workability, availability, resistance to deformation, and overall economy.

24. A wide range of materials may be used as ballast. Commonly used ballast materials include: crushed stone, crushed slag, prepared gravel, pit-run gravel, chat, cinders, chert, and sand. Crushed stone is probably the most available and widely used ballast material. Limestone, quartzite, basalt, and granite crushed to desired sizes varying from 3/4 to 3-1/2 in. are commonly used. These materials possess strength, durability, and stability and can be cleaned readily. Crushed steel mill or blast furnace slag makes excellent ballast possessing many of the characteristics of crushed stone. Prepared gravel or crushed pit-run gravel is frequently available at a relatively low cost; however, this ballast may not be cleaned. Chat, sometimes called screenings, are the tailings or refuse from the rock-crushing process. Pit-run gravel, chat, cinders, chert, and sand are inexpensive ballast materials lacking the quality of crushed stone, slag, and crushed gravels; however, these materials are often adequate for use on secondary, low-density tracks.

25. Theoretically the ballast section should be of sufficient depth to distribute the load from individual ties throughout the ballast to a uniform pressure acting on the subgrade. Early experimental work by Talbot (American Railway Engineering Association 1980) indicated that the lateral distribution of vertical pressures is nearly uniform at a depth approximately equal to the center-to-center tie spacing. Based on these results, the minimum depth of ballast recommended for main tracks is 24 in. when prepared stone ballast is used. The ballast section is considered to be failed if it does not provide the functions listed in paragraph 23. Characteristics of ballast materials which are believed to contribute to a loss in the function of the track support system are: (a) inadequate ballast thickness, (b) inadequate ballast resiliency, (c) ballast degradation, (d) ballast pumping, (e) ballast permanent deformation, and (f) fouled ballast (Robnett et al. 1975). If the ballast layer is not thick enough to properly reduce the stresses applied to the subgrade, excessive elastic deformation and rutting may occur contributing to other types of distress. If the ballast becomes cemented or loses its resiliency, undesirable dynamic loading effects may occur. Ballast degradation from mechanical breakdown, weathering, or chemical breakdown can result in the loss of the open-graded properties of ballast materials, possibly inducing cementing of the ballast. Ballast pumping occurs when the stresses at the ballast-roadbed interface are sufficient to cause the ballast and roadbed materials to

18

begin to intermix. As this process continues, the ballast becomes fouled and as additional ballast is placed on the track, it is forced into the roadbed forming a "ballast pocket." This pumping and ballast pocket formation inhibits the free-draining characteristics of the ballast. Permanent deformation of the ballast section occurs if the ballast section does not possess adequate stability and often results in loss of surface, line, and gage. Fouled ballast occurs when excessive fines contaminate the ballast material resulting in loss of free-draining properties and decreased shear strength. Ballast fouling occurs from ballast degradation, ballast pumping, or from fine material falling from rolling stock.

26. Subballast is a lesser quality ballast-type material placed under the ballast layer and over the roadbed. The primary functions of the subballast layer are to: (a) distribute the ballast pressure to the roadbed, (b) damp vibrations, (c) act as filter layer to prevent ballast fouling, and (d) protect the subgrade from water and frost penetration. Granular materials such as crushed stone, sand-gravel, crushed slag, stabilized soil, and in some cases hot-mix asphaltic concrete are used as subballast. The use of subballast in the track structure is optional; if present, the thickness will normally range from 6 to 12 in., but may vary depending on the existing conditions.

Industry standards

27. The type and amount of ballast used by commercial railroads vary with the type of track (main or secondary line), traffic type and tonnage, availability of ballast, cost of ballast materials, and subgrade conditions. Surveys of commercial railroads (Robnett et al. 1975) indicate that limestone, slag, granite, and pit-run gravel are the four most commonly used types of ballast materials. The ballast sizes and gradings for crushed stone, slag, and gravel recommended by the AREA are presented in Tables 2 and 3. "Railroad Engineering" (Hay 1982) states that the most popular grade of ballast for use is the AREA No. 4 ballast; however, with heavy axle loads, CWR, and concrete ties, larger ballast materials such as the AREA No. 24 are gaining favor.

Army standards and usage

28. Technical Manual 5-850-2/AFM 88-7, Chapter 2 (Headquarters, Departments of the Army and the Air Force July 1980) specifies a minimum ballast thickness of 8 in. with prepared stone, gravel, or slag as the preferred materials. The ballast materials should conform to AREA recommendations. In

areas where the roadbed is difficult to drain, a minimum of 6 in. of subballast may be used. The ballast section (dimensions and side slopes) should conform to the AREA recommendations. Figures 6 and 7 present typical ballast sections recommended by the AREA for tangent and curved track, respectively. Army Regulation 420-72 (Headquarters, Department of the Army March 1976) states
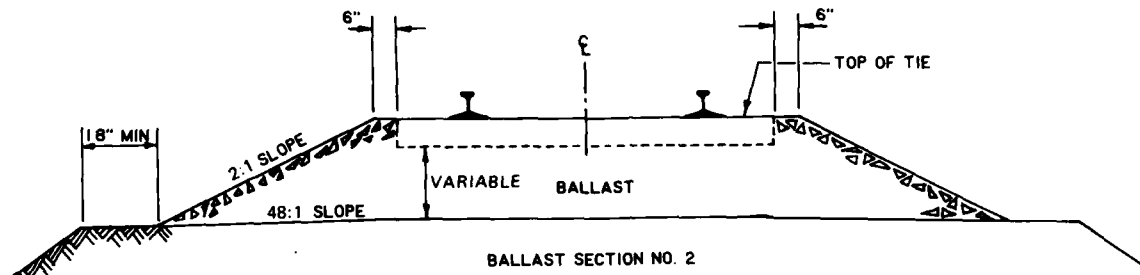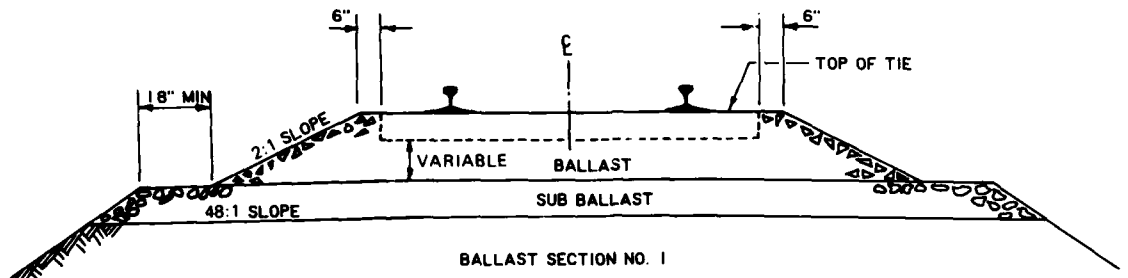

BALLAST SECTION NO. 1

BALLAST SECTION NO. 2

Figure 6. Typical ballast section, tangent track

BALLAST SECTION NO 1-A CURVES

BALLAST SECTION NO 2-A CURVES
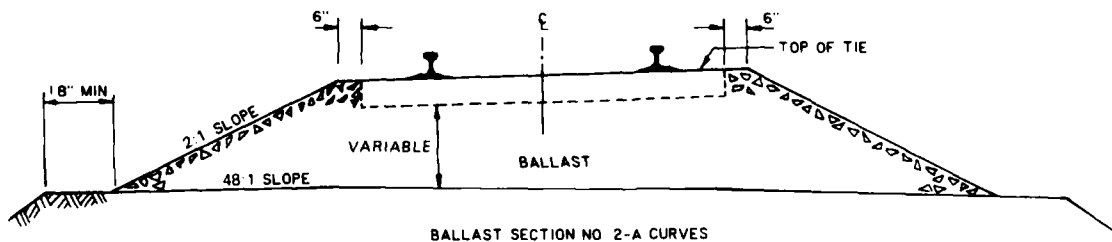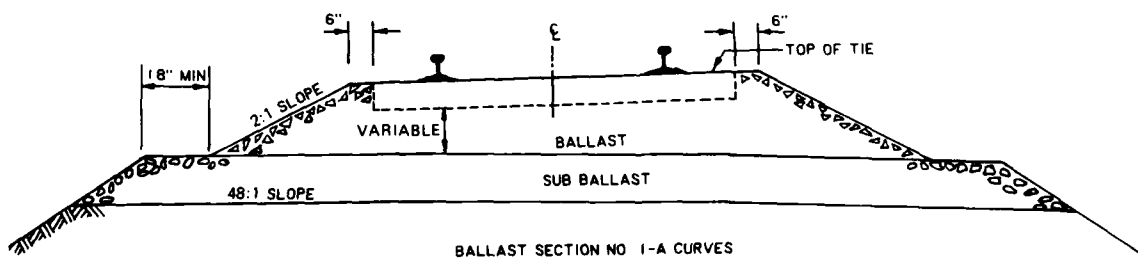
Figure 7. Typical ballast section, curved track

that stone, screened gravel, and slag ballast will be reconditioned, and cinders and pit-run gravel ballast will be replaced when dirt and other foreign material restrict proper drainage.

29. Ballast sections currently existing on installation tracks vary from nonexistent to very good and cover the entire spectrum between those extremes. As with ties, the ballast section and condition varies from installation to installation depending on track usage, maintenance funds, maintenance performed, and other factors.

Subgrade

30. The subgrade, or roadbed, is the prepared natural ground upon which the ballast section, ties, and rails are placed. The subgrade beneath railroad tracks performs three primary functions, namely: (a) supports and distributes the applied loads of locomotives and rolling stock, (b) facilitates drainage away from the track structure, and (c) provides a smooth platform on an established grade for the placement of the ballast, ties, and rails (Hay 1982).

31. The subgrade is composed of soil, a material with variable composition and variable performance. Weak or unstable soil conditions under railroad tracks, combined with the effect of water on soil strength, can result in a multitude of maintenance problems, high maintenance costs, and possibly train delay. The "Manual for Railway Engineering" (American Railway Engineering Association 1984), lists problems caused by subgrade conditions as: (a) pumping of subgrade soils into the ballast, (b) softening and squeezing of the subgrade soils in the presence of water, and (c) frost heaving of subgrade soils. When subgrade soils become saturated, repeated traffic loading may cause soil particles to be pumped up into the voids in the ballast causing the ballast to become fouled. Pumping occurs most frequently under poorly maintained rail joints, but may occur at any location along the track. An area where repeated track settlement occurs requiring frequent resurfacing is sometimes called a "soft spot." Soft spots usually occur where there are plastic subgrade soils, trapped water, and heavy traffic. Soft spots may develop into ballast pockets where ballast is forced deeper and deeper into the subgrade resulting in loss of surface and a continual maintenance problem. Roadbed frost heaving is caused by the simultaneous presence of fine-grained

21

material, water, and freezing temperatures. For detailed discussions of sub-
grade soils, subgrade construction, and roadbed instability problems, the
reader is referred to "Railroad Engineering" (Hay 1982) and the "Manual for
Railway Engineering" (American Railway Engineering Association 1984).

32. Subgrade conditions on both commercial railroads and installation
tracks are so varied that a description of existing conditions is impossible.
With day-to-day maintenance little can be done to improve the subgrade, except
to provide for adequate drainage. Providing adequate drainage to remove water
from the track structure is the most important maintenance activity that can
be performed to eliminate the subgrade problems described above. When new
construction or major track rehabilitation is performed, poor subgrade condi-
tions can often be eliminated by replacing the poor soil with a better quality
material and ensuring that adequate compaction is obtained over the new road-
bed. As in maintenance, providing adequate drainage to the track structure
should be a high priority during new construction.

PART III: RAILROAD DESIGN

33. Clarke (1957a) describes the basic problem in railroad track design as providing for the transfer of the forces produced by rolling wheel loads on the railhead to the roadbed while keeping the unit bearing pressure on the roadbed within safe limits. Railroad track design in the United States has, from the beginning, been mostly empirical, with track systems to a large extent being constructed instead of designed. This section reviews the design practices for conventional railroad track structures that are currently used in the United States and throughout the world.

AREA Design Method

34. The AREA "Manual for Railway Engineering" (American Railway Engineering Association 1984) provides the design criteria and analytic methods for railroad track design currently used in the United States and Canada. These criteria and methods were primarily developed by Professor A. N. Talbot and the AREA/ASCE (American Railway Engineering Association/American Society of Civil Engineers) Special Committee on Stresses in Railroad Track and published in seven progress reports between 1918 and 1942. These reports have been combined into one volume and republished by the AREA in 1980. The AREA design method is based on satisfying a number of design criteria for the strength of individual track components. These criteria include:

  a. Bending stress in the rail base.

  b. Tie bending stress.

  c. Pressure on the ballast surface under a tie.

  d. Pressure on the subgrade.

Beam-on-elastic-foundation analysis

35. The beam-on-elastic-foundation model is the key to the AREA design procedure. Kerr in "Problems and Needs in Track Structure Design and Analysis" (Kerr 1977) presents an outline of the development of this model for analysis of track structures. The fundamental differential equation which considers the track structure as a continuous, elastically supported beam is

23

$$EI \frac{d^4y}{dx^4} + Uy = 0 \tag{1}$$

where

E = rail modulus of elasticity = 30×10^6 psi

I = rail moment of inertia, in.^{4}

U = modulus of elasticity of the track support or track modulus, psi

y track deflection, in.

The track modulus, U , is defined by Talbot as "the pressure per unit length of each rail required to depress the track one unit" (American Railway Engineering Association 1980), and may be interpreted as "the load in pounds per lineal inch of rail which will depress the track one inch" (Clarke 1957a). Solution of Equation 1 to determine the rail deflection at any point resulting from a single point load yields

$$y(x) = \frac{P}{\sqrt[4]{64EIU^3}} e^{-\beta x}(\cos \beta x + \sin \beta x) \tag{2}$$

where

$y(x)$ = deflection at any distance x from the load, in.

P = design wheel loads, lb

U = track modulus, psi

e = 2.7183

x = distance from load to any point on the deflection and bending moment curves, in.

and

$$\beta = \left(\frac{U}{4EI}\right)^{1/4} \tag{3}$$

Taking successive derivatives of this deflection equation results in equations for calculating slope, bending moment, shear, and pressure intensity against the rail. The equation for calculating the bending moment at any location from the load, $M(x)$ in inch-pounds is

$$M(x) = P \sqrt[4]{\frac{EI}{64U}} e^{\beta x}(\cos \beta x - \sin \beta x) \tag{4}$$

The shear force in the rail at any point, $V(x)$, in pounds, is calculated from:

$$V(x) = -\frac{P}{2} e^{-\beta x} (\cos \beta x) \tag{5}$$

The pressure intensity against the rail, $p_r(x)$ in pounds per inch is defined as follows:

$$P_r(x) = -Uy(x) \tag{6}$$

or

$$P_r(x) = P \sqrt[4]{\frac{U}{64EI}} e^{-\beta x} (\cos \beta x + \sin \beta x) \tag{7}$$

Two other computations that are important in this analysis are the distance from the load to the point of zero bending moment, denoted X_1, and calculated using Equation 8, and the distance from the load to the point of contraflexure which is denoted X_2 and calculated using Equation 9.

$$X_1 = \frac{\pi}{4} \sqrt[4]{\frac{4EI}{U}} = 82.2 \sqrt[4]{\frac{I}{U}} , \text{ in.} \tag{8}$$

$$X_2 = \frac{3\pi}{4} \sqrt[4]{\frac{4EI}{U}} = 3X_1, \text{ in.} \tag{9}$$

The maximum deflection and maximum bending moment both occur at the point of load application $(x = 0)$. Solving Equation 2 for $x = 0$ results in the following equation for maximum deflection:

$$y_{max} = \frac{P\beta}{2U} = 0.391 \frac{P}{UX_1} \text{ (in.)} \tag{10}$$

In a like manner, the maximum bending moment, M_{max}, in the rail is

$$M_{max} = \frac{P}{4\beta} = 0.318 \ PX_1 \text{ (in.-lb)} \tag{11}$$

The maximum rail pressure also occurs at $x = 0$ and is found from

$$P_{r_{max}} = \frac{P\beta}{2} = 0.391 \frac{P}{X_1} \text{ (lb/in.)} \tag{12}$$

The rail seat load, q, on any individual tie is then

25

$$q = p_r s \qquad\qquad (13)$$

where

q = rail seat load, lb

p_r = pressure intensity against rail, lb/in.

s = tie spacing, in.

and the maximum rail seat load, q_o , is

$$q_o = 0.391 \frac{Ps}{X_1} \qquad\qquad (14)$$

The maximum value of bending stress in the rail, f_o , is

$$f_o = \frac{M_o}{Z} \qquad\qquad (15)$$

where

f_o = bending stress in base of rail, psi

M_o = maximum bending moment, in.-lb

Z = section modulus, in.^{3}

36. The values for the bending moment, deflection, and rail pressure resulting from a single-wheel load may be determined at any point along the rail in terms of the maximum values by using the Talbot master diagram presented in Figure 8. In this diagram the maximum values, which occur directly beneath the load, are taken as unity with values of bending moment, deflection, and pressure at points away from the load being expressed in terms of these maximum values. It must be pointed out that this master diagram is only for a single-wheel load. If more than one wheel is involved in the analysis the moment and deflection coefficients for each wheel should be determined using both the lower part of Figure 8 and the master diagram and composite deflection and bending moment values determined. The chart in the lower part of Figure 8 was developed using the principle of superposition. The steps given in Figure 8 are followed to determine the moment and deflection coefficients for each wheel falling within the range of influence. These coefficients are summed algebraically to determine the composite coefficient that is input into the appropriate moment or deflection equation.

Steps in the AREA design procedure

37. Figure 9 presents a flowchart for the AREA design of conventional

MASTER INFLUENCE CURVES FOR BENDING MOMENT, PRESSURE INTENSITY, AND TRACK DEPRESSION

p=INTENSITY OF PRESSURE (LB./LINEAR IN.)

RELATIVE VALUES OF BENDING MOMENTS (Cm)

RELATIVE VALUES OF TRACK DEPRESSIONS (Cd)

RELATIVE VALUES OF INTENSITY OF PRESSURES

A. DISTANCE TO POINT OF ZERO MOMENT, $X_1 = 82.2 \sqrt[4]{\frac{I}{u}}$

B. MAXIMUM BENDING MOMENT, $M_0 = (0.318\,P\,X_1)\,Cm$

C. MAXIMUM TRACK DEPRESSION, $y_0 = \left(\frac{0.39\,P}{u\,X_1}\right) Cd$

D. MAXIMUM INTENSITY OF RAIL PRESSURE $p_0 = u y_0$

BENDING MOMENTS IN RAIL, M_0
TRACK DEPRESSION, y_0
INTENSITY OF RAIL PRESSURE, p_0
COEFFICIENTS FOR

DISTANCE ALONG RAIL FROM LOAD POINT

VALUE OF X_1 IN INCHES

DISTANCE IN INCHES FROM LOAD POINT TO OTHER WHEELS

DESIGN STEPS

1. CALCULATE X_1 FROM FORMULA A ABOVE.
2. ENTER LOWER DIAGRAM WITH X_1, DRAW HORIZONTAL LINE TO INTERSECT LINE INDICATING DISTANCE TO ADJACENT WHEELS.
3. DRAW VERTICAL LINE TO INTERSECT UPPER GRAPH CURVES GIVING BENDING MOMENT AND DEPRESSION COEFFICIENTS.
4. READ COEFFICIENTS C_m AND C_d ON LEFT SCALE.
5. REPEAT FOR OTHER ADJACENT WHEEL LOADS AND ENTER TOTALS IN FORMULAS B AND C.

Figure 8. Master diagram for determining bending moment, pressure intensity and track depression (Adapted from "Manual of Railway Engineering" (American Railway Engineering Association 1984), used by permission.)

27
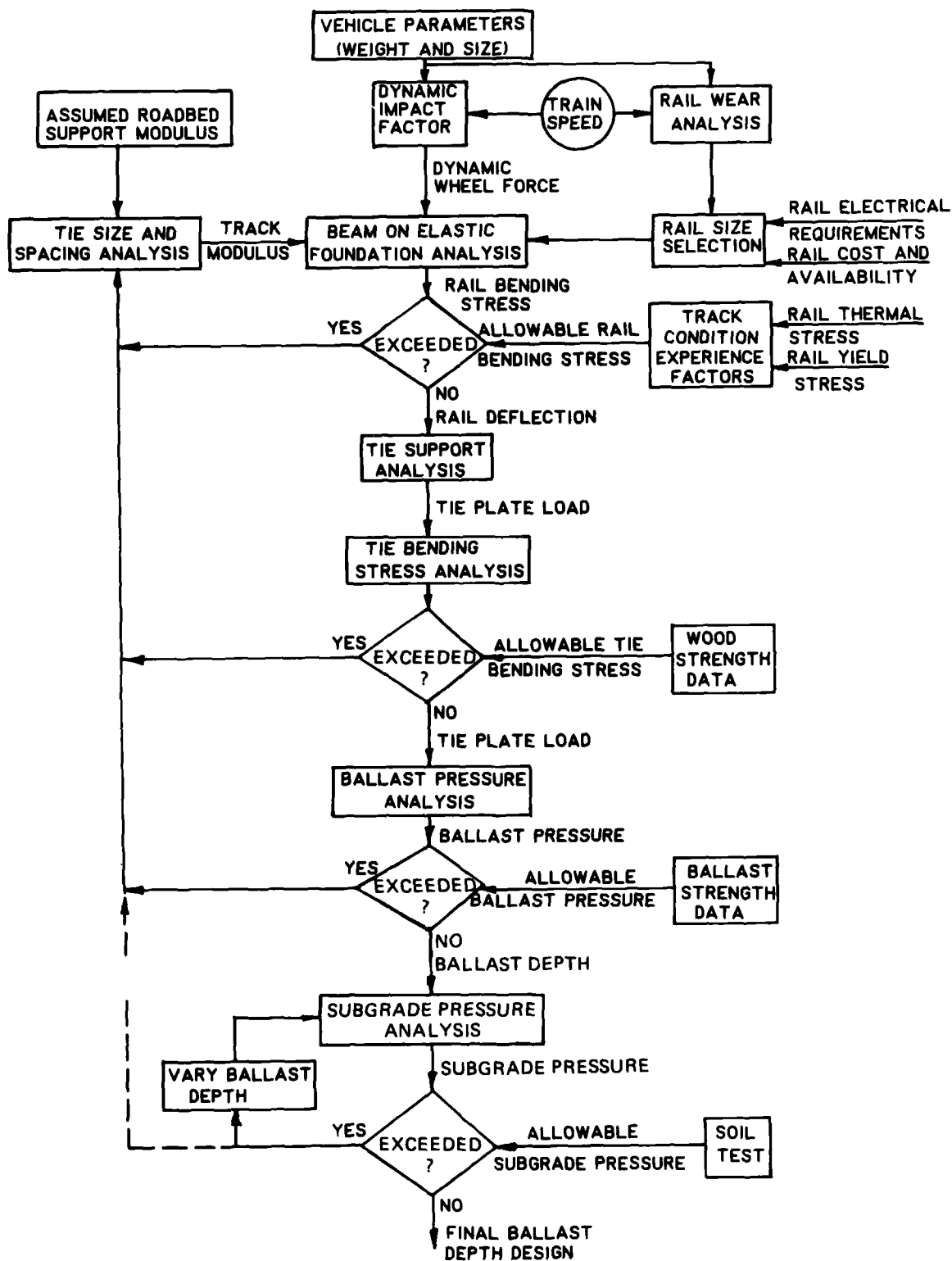
Figure 9. Flowchart for conventional track structure design

at-grade track structures. From this figure the principal steps in track design are:

a. Determine the design vehicle parameters and wheel loads.

b. Increase the wheel loads by the appropriate dynamic impact factor to determine the design wheel load.

c. Assume a track modulus for design.

d. Assume a tie size and center-to-center spacing.

e. Select a rail size.

f. Determine the allowable rail bending stress, $f_{o_{allow}}$

g. Calculate the maximum rail bending stress, f_o .

h. Check to assure that the maximum rail bending stress is less than the allowable rail bending stress. If not, select a larger rail size and repeat steps g and h.

i. Calculate maximum rail deflection, $y_{(max)}$.

j. Calculate maximum rail seat load, q_o .

k. Calculate the tie plate size required and choose the appropriate tie plate.

l. Calculate maximum tie bending stress, f_t .

m. Determine allowable tie bending stress, $f_{t_{allow}}$.

n. Check to assure that the maximum tie bending stress is less than the allowable tie bending stress. If not, select a larger tie size or smaller tie spacing and repeat steps e through n.

o. Calculate the average ballast pressure, p_b .

p. Check to assure that the maximum ballast pressure is less than or equal to 65 psi. If not, select a larger tie size or smaller tie spacing and repeat steps e through p.

q. Assume a ballast depth, h .

r. Calculate the maximum subgrade bearing pressure, p_s .

s. Determine the allowable subgrade bearing pressure, $p_{s_{allow}}$.

The allowable subgrade bearing pressure should not exceed 20 psi as recommended by the AREA.

t. Check to assure that the maximum subgrade bearing pressure is less than or equal to the allowable subgrade bearing pressure. If not, choose a thicker ballast depth and repeat steps r through t.

Each of these steps will be discussed in detail in the following paragraphs.

38. Design loads (steps a and b). The basic function of a track structure is to guide the train and to support the loads imparted by the train to

the track structure. The forces acting on the track structure are:

    a. Vertical loads due to the static weight of locomotives and rolling stock and vertical dynamic forces resulting from the cars' movement over deviations in track geometry.

    b. Lateral forces due to the cars' response to track deviations and external disturbances, forces from self-excited hunting motions, and forces necessary to guide the train through curves.

    c. Longitudinal forces due to traction, braking, and thermal expansion and contraction.

The actual forces transmitted to the track structure are very complex, and a detailed treatment of these forces is beyond the scope of this report. Detailed discussions of the forces acting on a track structure may be found in Railroad Engineering (Hay 1982), "Stresses in Railroad Track - The Talbot Reports" (American Railway Engineering Association 1980), "Assessment of Design Tools and Criteria for Urban Rail Track Structures (Prause et al. 1974), and "Ballast and Subgrade Requirements Study: Railroad Track Substructure - Design and Performance Practices" (DiPilato et al. 1983).

    39. The starting point for track design is the vertical force from the static weight of a railroad car or locomotive on the rails. Standard practice for track design in the United States is to use the maximum expected static wheel load increased by a speed dependent impact factor to determine the design wheel load. The impact factor recommended by the AAR, and presented in the AREA "Manual for Railway Engineering" (American Railway Engineering Association 1984), is

$$K = \frac{33V}{100D} \tag{16}$$

where

    V = speed, mph

    D = wheel diameter, in.

Clarke, in "Track Loading Fundamentals - Part 7, Various Speed Effect Formulae" (Clarke 1957c), compares various speed dependent impact factors and finds the AAR formula to yield impact factors that are about the median for those factors compared. For detailed discussions of speed impact factors used throughout the world, the reader is referred to "Track Loading Fundamentals - Part 7, Various Speed Effect Formulae" (Clarke 1957c), and "Railway Track Design: A Review of Current Practice" (Doyle 1980). The AREA recommends that the design wheel load for conventional wood-tie track be

$$P_d = (1 + K)P_s \qquad (17)$$

where

P_d = dynamic wheel load

K = impact factor from Equation 16

P_s = static wheel load

The maximum recommended dynamic wheel load is twice the static wheel load or

$$P_d = 2P_s \qquad (18)$$

40. Track modulus determination (step c). The track modulus has been previously defined as "the pressure per unit length of each rail required to depress the track one unit" (American Railway Engineering Association 1980). It represents the stiffness and yieldability of the tie, ballast, subballast, and subgrade, but does not include the rail stiffness. For conventional wood-tie track, values of track modulus can range from 400 psi on very poor track to 4,000+ psi on high-quality, well maintained track (Clarke 1957a). Well main-tained track having concrete ties may have track modulus values in the 7,000- to 8,000-psi range (Hay 1982). The AREA recommends a 2,000-psi track modulus for use in design. However, the actual value will be influenced by tie quality, size, and spacing; ballast and subballast thickness and density; and subgrade strength. Determination of track modulus on existing track from load-deflection measurements will be discussed in Part VI of this report. In the course "Railroad Tracks Design, Analysis, and Maintenance," A. D. Kerr* recom-mends that three track modulus values be used for design in order to bound the problem: the smallest expected track modulus for determining the maximum rail bending moment, the average expected track modulus for determining the ballast depth, and a largest expected track modulus for determining the rail pressure and rail seat load.

41. Select tie size and spacing (step d). Tie sizes and spacings for use in Army track are specified in TM 5-850-2 AFM 88-7, Chapter 2 (Head-quarters, Department of the Army 1980), as outlined in paragraph 21 of this report. The tie sizes and spacings used for the design of Army track should conform to these specifications.

42. Select rail size (step e). Selection of rail size is based primar-ily on providing sufficient bending capacity to keep the bending stresses

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31

within safe limits. In commercial railroad practice other factors such as electrical requirements, current and future availability from rail suppliers, and cost also enter into the selection process. As a guide for determining the minimum required rail weight, the "Manual for Railway Engineering" (American Railway Engineering Association 1984) provides three equations, any one of which may be used as a starting point in rail size selection. These equations are

$$W_o = 156 - \frac{10,600}{(2P)(\alpha) + 67)} \qquad (19)$$

where

W_o = weight of rail, kg/m

$2P$ = axle load, metric tons

α = impact factor, usually 2

$$W_o = 17 \left(\frac{P + 0.0001PV^2}{2,000} \right)^{2/3} \qquad (20)$$

where

W_o = weight of rail, lb/yd

P = static wheel load, lb

V = train velocity, mph

$$Z_b = \frac{0.318P_d X_1}{f_o} \qquad (21)$$

where

Z_b = section modulus of rail base, in.^{2}

P_d = dynamic wheel loads, lb

X_1 = distance from wheel to point zero bending moment, in.

f_o = maximum flexural stress in the base of rail, psi

Any one of these formulae may be used to determine the minimum rail section to use in the first design trial; however, a greater rail weight may be required based on the rail stresses, rail wear, or other considerations. The AREA suggests that each equation be used as a cross-check on the other two (American Railway Engineering Association 1984).

32

43. Allowable rail bending stress determination (step f). The AAR (American Railway Engineering Association 1984) recommends that the maximum allowable bending stress for rail be calculated as follows:

$$\sigma_{all} = \frac{\sigma_y - \sigma_t}{(1 + A)(1 + B)(1 + C)(1 + D)} \tag{22}$$

where

σ_{all} = allowable bending stress, psi

σ_y = yield stress of the rail steel, psi

σ_t = temperature-induced stress in the rail = 7,000 psi

A = stress factor to account for lateral bending of the rail = 20 percent

B = stress factor to account for track conditions = 25 percent

C = stress factor to account for rail wear and corrosion = 15 percent

D = stress factor to account for unbalanced superelevation of track = 15 percent

Assuming a rail steel yield strength of 70,000 psi, Equation 22 gives an allowable bending stress, σ_{all} , of

$$\sigma_{all} = \frac{70,000 - 7,000}{(1.20)(1.25)(1.15)(1.15)} = 31,758 \text{ psi}$$

Essentially, this equation reduces the yield stress by 7,000 psi for temperature-induced stresses and then divides the remainder by a factor of 2 to account for lateral bending, track conditions, rail wear, and corrosion and unbalanced superelevation. For CWR the temperature stress reduction is 20,000 psi resulting in σ_{all} equal to 25,000 psi.

44. Maximum bending stress calculation (steps g and h). The maximum bending stress, f_o , in the rail is calculated using Equation 15. If the maximum calculated rail bending stress is greater than the allowable rail bending stress, a larger rail size should be selected and the bending stress calculation repeated.

45. Calculate the maximum rail deflection (step i). The maximum rail deflection, y_{max} , is calculated using Equation 10. The "Manual for Railway Engineering" (American Railway Engineering Association 1984) recommends that

33

the track depression be limited to 0.25 in. in order to maintain good ride
quality and to reduce maintenance costs. If the maximum calculated rail de-
flection exceeds the 0.25-in. maximum deflection maintenance criterion, Kerr
recommends that a heavier rail weight be chosen in order to reduce the maximum
deflection.

46. Calculate the maximum rail seat load (step j). Maximum rail seat
load, q_o , is calculated using Equation 14.

47. Calculate required tie plate size and choose appropriate tie plate
(step k). The required tie plate size may be calculated by dividing the maxi-
mum rail seat load, q_o , by the allowable tie compressive stress as shown in
Equation 23:

$$A_p = \frac{q_o}{\sigma_{t_{allow}}}$$
(23)

where

A_p = area of tie plate required, in.^{2}

q_o = maximum rail seat load, lb

$\sigma_{t_{allow}}$ = allowable tie compressive stress, psi

Table 4 presents the recommended allowable tie compressive stress for various
types of wood in the continuously dry and wet conditions, extracted from
"Manual for Railway Engineering" (American Railway Engineering Association
1984). Kerr recommends the allowable tie compressive stress used in design be
in the range of 100 to 150 psi. After calculating the tie plate AREA required,
the appropriate plate may be chosen from Chapter 5-1-7 of the "Manual for
Railway Engineering" (American Railway Engineering Association 1984).

48. Calculate the maximum tie bending stress (step l). The maximum tie
bending stress, f_t , is calculated from

$$f_t = \frac{6q_o}{A_b} \frac{Q^2}{t^2}$$
(24)

where

f_t = maximum tie bending stress, psi

q_o = maximum rail seat load, lb

A_b = tie bearing area

Q = distance from center of rail seat load to the end of tie, in.

t = tie thickness, in.

The tie bearing area, A_b , is calculated using

$$A_b = bL \tag{25}$$

where

b = tie width, in.

L = effective tie bearing length, in.

Clarke (1957b) recommends computing L for wooden ties using the formula:

$$L = 2Q \left(1 - \frac{0.036Q}{t^{3/4}}\right) \tag{26}$$

where

L = effective tie bearing length, in.

Q = distance from center of rail seat load to end of tie, in.

t = tie thickness, in.

For the normal sizes and standard gage track used in the United States, Equation 26 can be closely approximated by

$$L = \frac{\ell}{3} \tag{27}$$

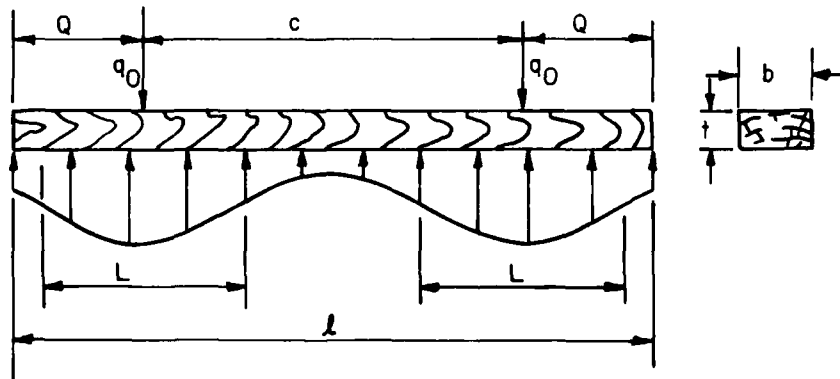where ℓ equals the tie length as shown in Figure 10 (Prause et al. 1974).

Figure 10. Typical pressure distribution
along tie length

49. Determine allowable tie bending stress (steps m and n). The allowable tie tensile stress varies for different wood species as indicated in

Chapter 7-1 of "Manual for Railway Engineering" (American Railway Engineering Association 1984); however, the AREA recommends an allowable tensile stress of 1,100 psi for design (American Railway Engineering Association 1984, Chapter 22-3). If the *maximum* tie bending stress, f_t, is larger than the recommended 1,100 psi, a different tie size and/or spacing should be chosen and steps d through n repeated.

50. Calculate the average ballast pressure (steps o and p). The average ballast pressure rather than the maximum pressure is used to calculate the required ballast depth. Equation 28 is used to estimate the average ballast pressure, p_b, directly beneath the tie:

$$p_b = \frac{q_o}{A_b} \qquad (28)$$

where

p_b = average ballast pressure, psi

q_o = maximum rail seat load, lb

A_b = effective tie bearing area, in.$^2 \approx \frac{\text{tie length}}{3} \times$ (tie width) from paragraph 48

The recommended limit for average ballast pressure on wood-tie track is 65 psi (American Railway Engineering Association 1984), although values as low as 35 psi have been recommended (Clark 1957a) to prevent crushing of the ballast particles. The AREA "Manual for Railway Engineering" (1984) recommends on page 22-3-15 doubling the rail seat loads to account for increased pressures resulting from play between the rail and tie, variations in ballast tamping, and variations in roadbed strength. This doubling was first recommended by the Talbot Committee (American Railway Engineering Association 1980); however, this increase is taken care of, and doubling of the rail seat load is not required when the static load is doubled to account for dynamic effects as outlined in paragraph 39. If the average ballast pressure calculated from Equation 28 is greater than 65 psi, a closer tie spacing is chosen and steps d through p are repeated.

51. Assume a ballast depth (step q). A first trial ballast depth may be assumed as the center-to-center tie spacing. Experiments by the Talbot Committee (American Railway Engineering Association 1980) indicated that the vertical pressure on the subgrade is approximately uniform at a depth approximately equal to the center-to-center tie spacing. Hay (1982) recommends a

36

ballast depth equal to the tie spacing as a desirable minimum.

52. Calculate maximum subgrade bearing pressure (step r). The maximum subgrade bearing pressure, p_s , may be calculated using any one of the Equations 29 through 32 given below:

Talbot equation:

$$p_s = \frac{16.8 p_b}{h^{1.25}} \tag{29}$$

Japanese National Railways (JNR) equation:

$$p_s = \frac{50 p_b}{10 + h_m^{1.35}} \tag{30}$$

Boussinesq equation:

$$p_s = \frac{6 q o}{2 \pi h^2} \tag{31}$$

Loves equation:

$$p_s = p_b \left[1 \left(- \frac{1}{1 + \frac{r^2}{h^2}} \right)^{3/2} \right] \tag{32}$$

where

p_s = maximum subgrade bearing pressure, psi

p_b = average ballast pressure calculated from Equation 28, psi

h = ballast thickness, in.

h_m = ballast thickness in JNR equation, cm

$q o$ = maximum rail seat load, lb

r = radius of a uniformly loaded circle whose area equals the effective tie bearing area under one rail seat, A_b , in.

Both Equations 29 and 30 were developed empirically. The JNR equation was developed for narrow gage track and should not be used for standard gage track. The Talbot equation (Equation 29) was developed from a number of full-scale laboratory tests on various ballast materials including sand, slag, crushed stone, and gravel.

37

Equations 31 and 32 are both based on the Boussinesq solution for stress in an elastic body due to an applied surface point load (DiPilato et al. 1983). "Railway Engineering (Hay 1982) recommends the Talbot equation for calculating the subgrade pressure.

53. Determine the allowable subgrade bearing pressure (step $\underline{s}$). The recommended maximum allowable subgrade bearing pressure, $p_{s_{allow}}$, is 20 psi (American Railway Engineering Association 1984). While this value is a recommended design value, the actual allowable subgrade bearing pressure may be considerably different. Clarke in "Track Loading Fundamentals - Part 3" (1957b) recommends that the allowable bearing pressures on roadbed under track be limited to 60 percent of the normal allowable soil bearing capacity to account for variations in values of tie support with a maximum average subgrade pressure of 12 psi for uncompacted roadbed and 20 psi for compacted roadbeds. Table 5 presents typical allowable average subgrade bearing pressures as reported by Clarke (1957b); Headquarters, Department of the Navy (1971); and Milosevic (1969); and summarized by DiPilato et al. (1983). The allowable bearing pressures taken from the NAVFAC DM-7 have been multiplied by 0.6 as recommended by Clarke (1957b) and described above. Hay (1982) recommends that the Talbot value of subgrade bearing pressure, calculated from Equation 29, not exceed the ultimate bearing capacity of the subgrade soil. He defines the ultimate bearing capacity, q_d, as

$$q_d = 2.5q_u \tag{33}$$

where

$\quad q_d$ = ultimate soil bearing capacity, psi

$\quad q_u$ = unconfined compressive strength, psi

Including a 50 percent factor of safety, Equation 33 becomes

$$q_d = \frac{2.5q_u}{1.5} = 1.67q_u \tag{34}$$

Therefore, the maximum subgrade bearing pressure should be less than or equal to $1.67q_u$ as seen below:

$$p_s \leq 1.67q_u \tag{35}$$

Hay (1982) also states that the 50 percent factor of safety should be a minimum with a larger factor of safety used when the design engineer deems it necessary. Ireland (1973) states that a factor of safety of at least 2 is desirable. Although economics will sometimes dictate a lower factor of safety, it should not be less than 1.5 unless a certain amount of creep and deformation are acceptable. The unconfined compressive strength of a cohesive subgrade soil may be obtained from laboratory testing of undisturbed soil samples as detailed in ASTM D 2166 (American Society for Testing and Materials (ASTM) 1983). The unconfined compression test is a type of triaxial test that is very similar to the standard compression test performed on concrete cylinders. A sample of cohesive soil is trimmed to the specified dimensions, placed in the loading device without any confining membrane or confining pressure as in a triaxial test, and loaded from the top until failure occurs as indicated by a decrease in load resistance or by excessive strain in plastic materials (Krebs and Walker 1971). Expedient methods of determining unconfined compressive strength are the cone penetrometer and the pocket penetrometer.

54. Chapter 1 of the "Manual for Railway Engineering" (American Railway Engineering Association 1984) outlines the site investigation and materials testing techniques for use in determining the physical properties of the track subgrade. No detailed test procedures are given in the AREA Manual as all the tests are performed in accordance with ASTM standards. Detailed descriptions of the various site investigations and materials testing methods are presented by Simon, Edgers, and Errico in "Ballast and Subgrade Requirements Study Railroad Track Substructure - Materials Evaluation and Stabilization Practices" (1983), as well as in the indicated ASTM standards.

55. Check allowable subgrade bearing pressure (step $\underline{t}$). The maximum subgrade bearing pressure, p_s, calculated in step $\underline{r}$ should be less than or equal to the allowable subgrade bearing pressure, $p_{s_{allow}}$, determined in step $\underline{s}$. If p_s is greater than $p_{s_{allow}}$ the subgrade will be overstressed. A new ballast depth should be assumed and steps $\underline{q}$ through $\underline{t}$ repeated until the maximum subgrade bearing pressure is less than the allowable subgrade bearing pressure.

56. The steps for track design presented in the previous paragraphs are found in various sections of the "Manual for Railway Engineering" (American Railway Engineering Association 1984); however, a step-by-step procedure as

39

outlined in paragraph 37 is not given in the AREA Manual. Various authors including DiPilato et al. (1983) and Robnett et al. (1975) indicate that the major United States railroads have standard track designs for various regions of their operational areas. Most of these standard track designs are based on experience with little or no reference to theory. DiPilato et al. (1983) reported interviews with practicing railroad engineers indicating that in many cases little or no subsurface investigation is carried out to determine subgrade soil properties and allowable strengths prior to track construction or rehabilitation.

Foreign Design Practice

57. DiPilato et al. in "Railroad Track Substructure - Design and Performance Evaluation Practices" (1983) described the current substructure analysis and track design methods used by railroads in Great Britain, Japan, West Germany, Czechoslovakia, Hungary, and India. The principal design criterion used in each of these countries is to limit the resilent subgrade stresses to a level that will prevent bearing capacity failure and will limit the amount of permanent subgrade settlement. This is similar to current practice in the United States; however, the required ballast and subballast thicknesses are determined by practical analytical and experimental methods taking into consideration the physical properties of the subgrade materials.

British Railways design method

58. The British Railways design method described by Heath et al. (1972) is a rational, empirical, and analytical based method for determining the thickness of higher quality material (ballast and subballast) above clay subgrade materials for nonjointed (CWR) track. The basic design criterion is to protect against subgrade failure resulting from excessive residual deformation by limiting the amount of resilent stress and resilent strain in a clay subgrade to less than a limiting "threshold" stress. This limiting threshold stress is determined from the standard laboratory cyclic load triaxial test on undisturbed samples of the clay subgrade. In this test the radial principal stresses are maintained at a constant confining pressure typical of the pressure conditions under the track while a constant amplitude square-wave loading is applied at a constant rate of 30 cycles/min. The threshold stress is defined as the resilent stress level above which the soil deformation is very

40

rapid and below which the deformation accumulation rate is very slow. Ten percent cumulative strain after 10,000 cycles is often the limit used for determining the threshold stress. Heath et al. (1972) reports the four main assumptions inherent in this design method as:

a. The threshold stress parameters for a subgrade soil may be obtained using the standard repeated load triaxial test.

b. Simple elastic theory can be used to predict the stresses in the subgrade from traffic loading.

c. The significant traffic stresses are the stresses produced by the static effect of the heaviest commonly occurring axle load.

d. The water table is at the top of the subgrade.

59. The basis of this procedure is to equate the threshold shear stress determined from the laboratory testing with the shear stress computed beneath the ties using the Boussinesq distribution and the heaviest commonly occurring axle load to achieve a balanced design. In this balanced design the ballast/subballast layer is sufficiently deep so that the calculated maximum principal stress difference induced in the subgrade by the heaviest commonly occurring axle load is equal to the average threshold principal stress difference established by laboratory tests. The theoretical subgrade maximum principal stress differences for various axle loads is plotted and the threshold stress-depth relationship is superimposed on these curves to produce Figure 11. The intersection of the threshold stress/depth relationships and the maximum principal stress differences for the various axle loads yield the depth of ballast at which the subgrade threshold stress is equal to the stress produced by the given axle load. These intersection points can then be plotted to produce the design curves in Figure 12. These design curves are developed for particular track structures of a specified rail size, tie type, tie size, and tie spacing.

60. Laboratory tests and field track measurements to assess this design method indicated that reduced settlement rates were consistently achieved when the ballast thickness equaled or exceeded the design depth. When the ballast depth was less than the design depth, the settlement rates were significantly higher.

61. Several unsolved problems with this design method that have been recognized are:

a. Using the heaviest commonly occurring static axle load without correcting for dynamic effects may not be valid.

41
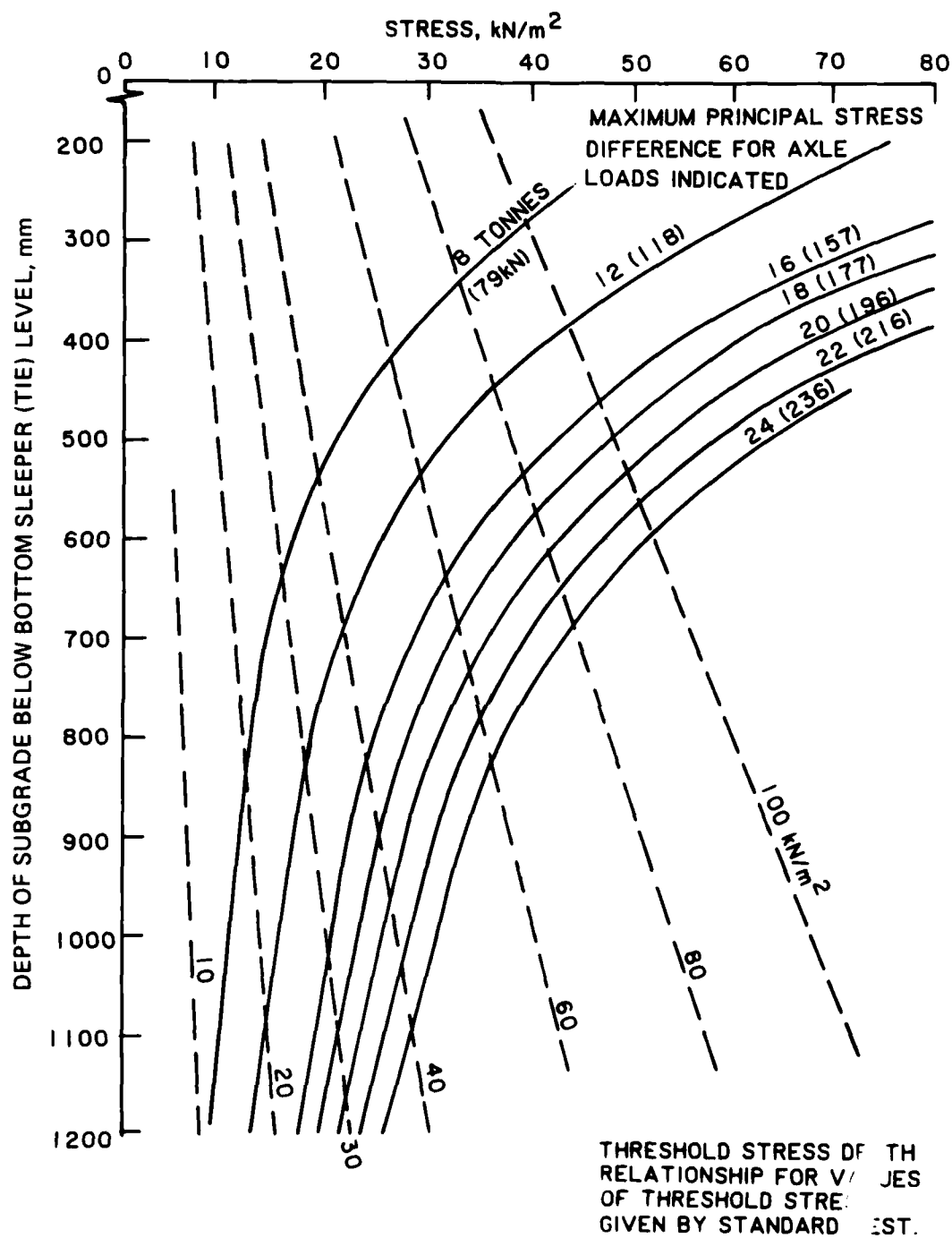
Figure 11. Derivation of British Railway design chart;
relationship between induced stresses and soil strength
(after Heath et al. 1972; used by permission)

THRESHOLD STRESS FROM STANDARD TEST $(\sigma_1 - \sigma_2)$ kN/m^2

DEPTH OF CONSTRUCTION, mm
BOTTOM OF SLEEPER TIE TO TOP OF SUBGRADE

DESIGN AXLE
LOAD
TONNES   (kN)

8   (76)
12   (118)
16   (157)
20   (196)
22   (216)
24   (235)

NOTE: CURVES ARE FOR A SPECIFIC
RAIL SIZE, TIE TYPE, AND
TIE SPACING.
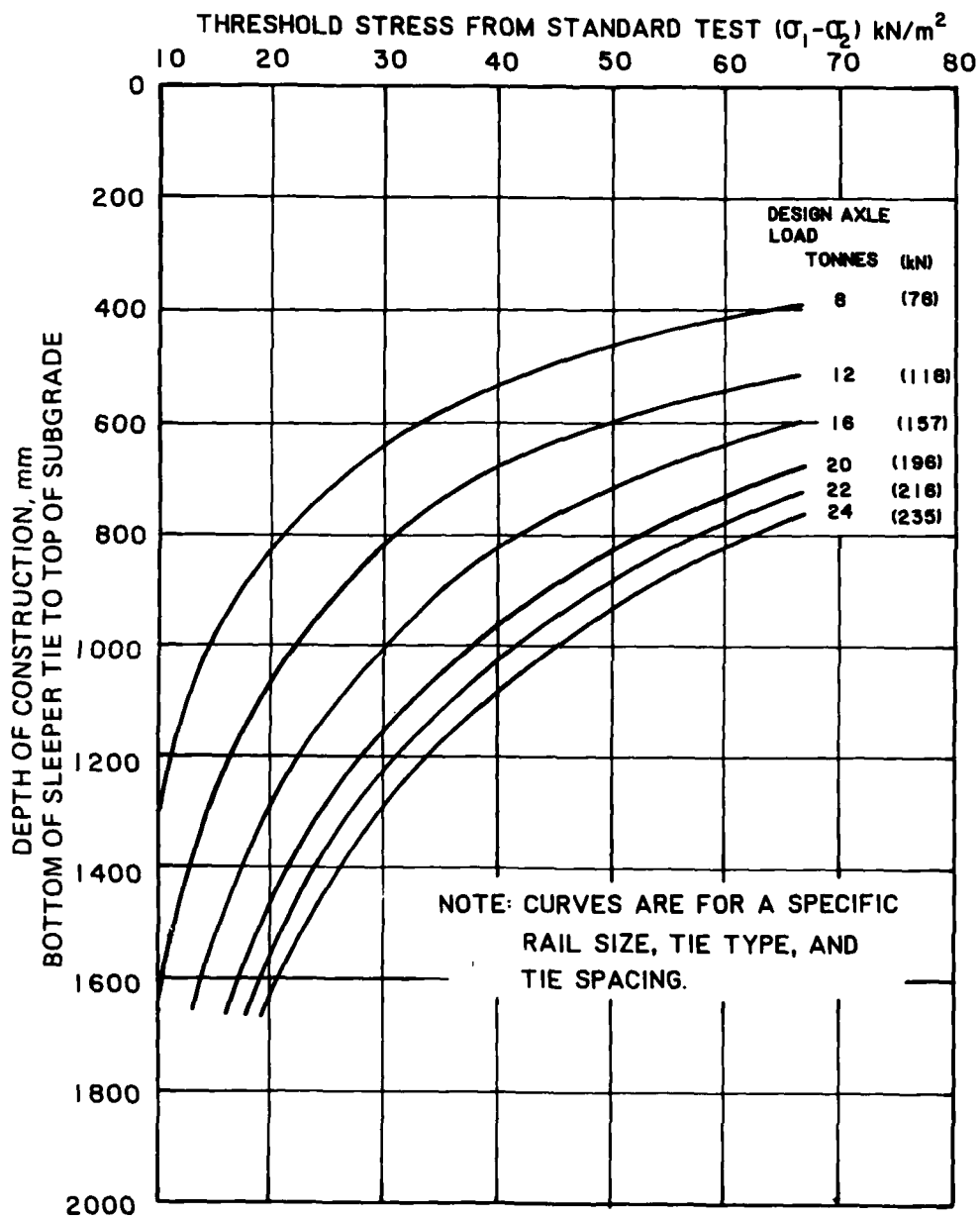
Figure 12.   Typical British Railway design chart
(after Heath, et al. 1972; used by permission)

43

<u>b</u>. The design loading does not consider the number of load cycles at or above the design load. Where the track has very high proportions of axle loads near the design load overstressing of the subgrade may occur.

<u>c</u>. The design procedure is applicable only to a stiff clay and its applicability to other clay subgrades or granular soils has not been determined.

Japanese National Railways (JNR) method

62. DiPilato et al. (1983) reports that the JNR is attempting to develop a maintenance free track structure for cohesive subgrades. The JNR design method is an empirical method using railroad experience combined with a multilayer flexible pavement design approach. The principal design criterion is limiting the stress on the subgrade to limit the residual displacement. The standard California Bearing Ratio (CBR) test is used to determine the deformation properties of the subgrade. A standard 10-in. ballast section is used. Additional strength is provided, where required, by varying the thicknesses of a crushed stone subballast. Standard track sections have been developed based on the subgrade CBR as shown in Table 6. Drainage systems to intercept, collect, and remove surface runoff and ground water from the track structure are a standard part of the JNR design method. Figure 13 presents a cross-sectional view of a typical JNR track structure.
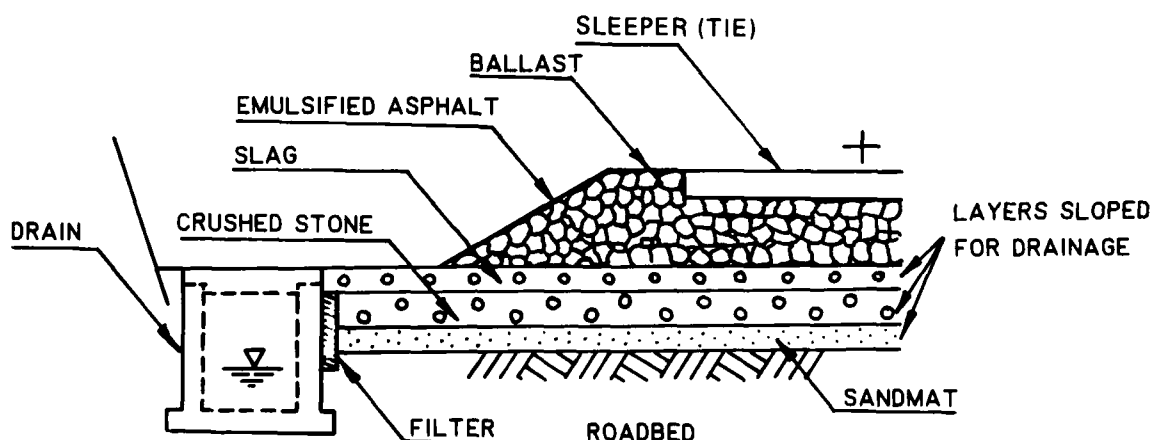
Figure 13. Typical JNR track structure (after DiPilato et al. 1983; used by permission)

63. The analysis method used to determine the required combined subballast thickness to reduce the vertical stress on the subgrade is presented in

44

Figure 14. The rail wheel load is converted to a "distribution load" as shown in Figure 14a, which is equivalent to the highway wheel loads given in Figure 14b. The design chart in Figure 14c is entered with the wheel load and appropriate CBR to determine the combined thickness of subballast.

b+2d=0.2 + 0.25 x 2=0.7m   0.7 x 2.6=1.82m^2

a+2d=2.1 + 0.25 x 2=2.6m   RAILWAY LOAD: KS-18=18t

d=BALLAST DEPTH=0.25m   TRACK LOAD=2.05t

                        IMPACT=0.5 18 x 0.5=9t

$$\text{DISTRIBUTED LOAD} = \frac{18+9+2.05}{1.82}$$

$$= 15.97 t/m^2$$
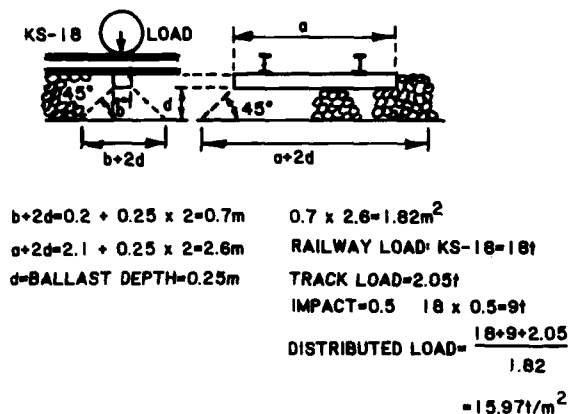
a. EQUIVALENT RAILROAD DISTRIBUTION LOAD

0.2 x 0.8=0.16m^2

$$\text{DISTRIBUTED LOAD} = \frac{P}{0.16} \, t/m^2$$

HIGHWAY LOAD/DISTRIBUTED LOAD

A=P=3t =19t/m^2

B=5t =31

C=8t =50

D=12t =75
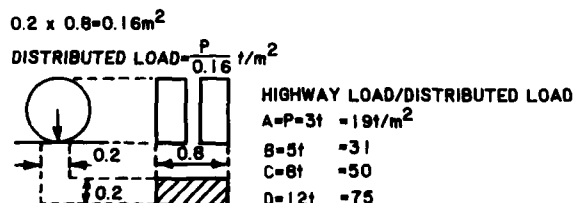
b. HIGHWAY WHEEL LOAD
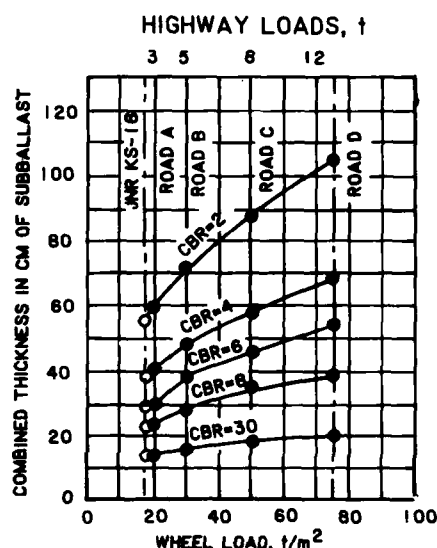
c. CHART FOR DETERMINING COMBINED SUBBALLAST THICKNESS (FROM FLEXIBLE PAVEMENT DESIGN)

Figure 14. JNR method to determine combined subballast thickness
(after DiPilato et al. 1983; used by permission)

64. DiPilato et al. (1983) reports that the JNR has found that this multilayer system provides improved shear strength, less settlement, improved frost protection, resilience under repeated loads, improved vibration damping, and improved subsurface drainage. Further field studies of in-service track are currently being conducted to evaluate long-term performance under various environmental conditions and vertical loads in order to determine the long-term maintenance requirements.

West German, Czechoslovakian, and
Hungarian State Railway design methods

65. Several authors who have reported on European railroad design have

been summarized in "Railroad Track Substructure - Design and Performance Evaluation Practices" (DiPilato et al. 1983). The design and analysis methods used by the German Federal Railway (DB), Czechoslovakian State Railways (CSD), and the Hungarian State Railways (MAV) are similar in that they:

a. Are empirical methods based on highway flexible pavement design procedures that used the elastic properties of the various layers to determine the thickness of the ballast/subballast layers.

b. Have the basic design criterion of limiting the stresses on the subgrade to those that can be supported with limited settlement.

c. Use geotechnical engineering methods to evaluate the type, strength, and elastic properties of subgrade materials.

d. Employ standard ballast thicknesses with additional layers of high-strength materials to provide additional load distribution to the subgrade.

e. Choose standard substructure sections based on subgrade type and strength.

f. Rely on experience gained from quantitative observations of substructure performance after construction or rehabilitation to evaluate substructure designs.

g. Provide high quality drainage systems to promote removal of surface and subsurface water.

66. The design approaches used by the DB, CSD, and MAV determine the required ballast and subballast thickness from the allowable pressure on the subgrade using a trial-and-error approach in which:

a. Ballast and subballast thicknesses are selected on experience. The DB and CSD use a standard ballast thickness of 12 in., while the MAV uses a 20-in. standard ballast thickness.

b. The elastic moduli of the various layers is used to determine an equivalent modulus for the entire substructure.

c. The vertical stress with depth below the tie is determined using single-layer elastic theory empirically modified by the different railroads.

d. The vertical subgrade stress is compared with the allowable subgrade stresses determined for the track. If the vertical stress is greater than the allowable subgrade stress, the protective layer thickness is increased and the vertical stress calculation is repeated.

67. The DB uses both the plate load test and CBR test to evaluate the subgrade deformation properties. The plate load tests are performed during the spring thaw or worst seasonal condition in order to consider environmental factors in the design. Traffic levels are considered in the design by

46

requiring a higher track stiffness for heavier traffic lines. Evaluation of the substructure is performed using multilayer elastic layer analysis techniques.

68. The CSD uses detailed subsurface explorations including visual observations, test borings, and test pits from which soil samples are taken for density and consistency tests. A standardized 12-in.-diam plate load test is used to determine the deformation properties of the subgrade materials.

69. The CBR test is used by the MAV for evaluating subgrade strength and deformation characteristics. Based on their experience, the MAV has established minimum subgrade CBR requirements for use with a 20-in. ballast layer. A 20-in. ballast layer without any subballast requires a miminum CBR of 14, while a 20-in. ballast with a 12-in. subballast layer requires a minimum CBR of 6.

70. Based on the above design procedures, total ballast/subballast thickness requirements typically range from 24 to 34 in. for the MAV, from 16 to 32 in. for the CSD for a 22,000-lb static wheel load, and from 22 to 34 in. for the DB for a 22,000-lb static wheel load.

Indian State Railways method

71. The Indian State Railways method was first reported by Agarwal and Yog of the Indian State Railways in 1975 and is summarized by DiPilato et al. (1983). This track design method is based on calculations of track substructure stresses using elastic methods (the Boussinesq equations) and evaluation of the allowable subgrade stress using the effective stress, Mohr-Coulomb failure model. The method was developed for a 5-ft, 6-in. rail spacing and 22.5-ton axle load; however, DiPilato indicates that the procedure is applicable to other track structures. The live load vertical and horizontal stresses caused by the train loads are computed from the elastic distribution with the tie seat represented as a rectangular footing about one third the tie length. The horizontal stresses are determined from the calculated vertical stresses using the coefficient of lateral earth pressure at rest, K_o. Excess pore pressures resulting from the live loads are evaluated using Skempton's pore pressure equation:

$$\Delta u = B \left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \right] \tag{36}$$

where

Δu = excess pore pressure

B = pore-water pressure parameter that is 1.0 for full saturation conditions and zero for a dry soil

$\Delta\sigma_3$ = increase in minor principal stress (lateral stress)

$\Delta\sigma_1$ = increase in major principal stress (vertical stress)

A = pore-water pressure parameter that depends on the type of clay, shear stress, and magnitude of strain

The excess pore pressures are then used to calculate the effective subgrade vertical and lateral stresses, $\bar{\sigma}_v$ and $\bar{\sigma}_h$, using:

$$\bar{\sigma}_v = \sigma_v - u_t \qquad (37)$$

$$\bar{\sigma}_h = \sigma_h - u_t \qquad (38)$$

$$u_t = u_s + \Delta u \qquad (39)$$

where

$\bar{\sigma}_v, \bar{\sigma}_h$ = effective vertical and lateral stresses at some depth

σ_v, σ_h = total (static plus dynamic) vertical and lateral stresses at that depth

u_t = total pore pressure at that depth

u_s = static pore pressure before loading

Δu = excess pore pressure due to loading

The shear strength of the subgrade is determined from laboratory testing on saturated samples. The results of this laboratory testing are plotted along with the effective stress state envelope as shown in Figure 15. The intersection of the failure envelope and effective stress envelope yields the depth of combined ballast and subballast required to prevent overstressing the subgrade. This design is intended to prevent subgrade failure. A second type of failure mechanism is subgrade pumping. To reduce the occurrence of subgrade pumping, the Indian State Railways recommends placement of a granular sand filter layer. To meet both the strength criteria and filter criteria, two subballast layers may be required.

48

INDIAN (BG) GAGE TRACK, 22.5 TON AXLE LOAD
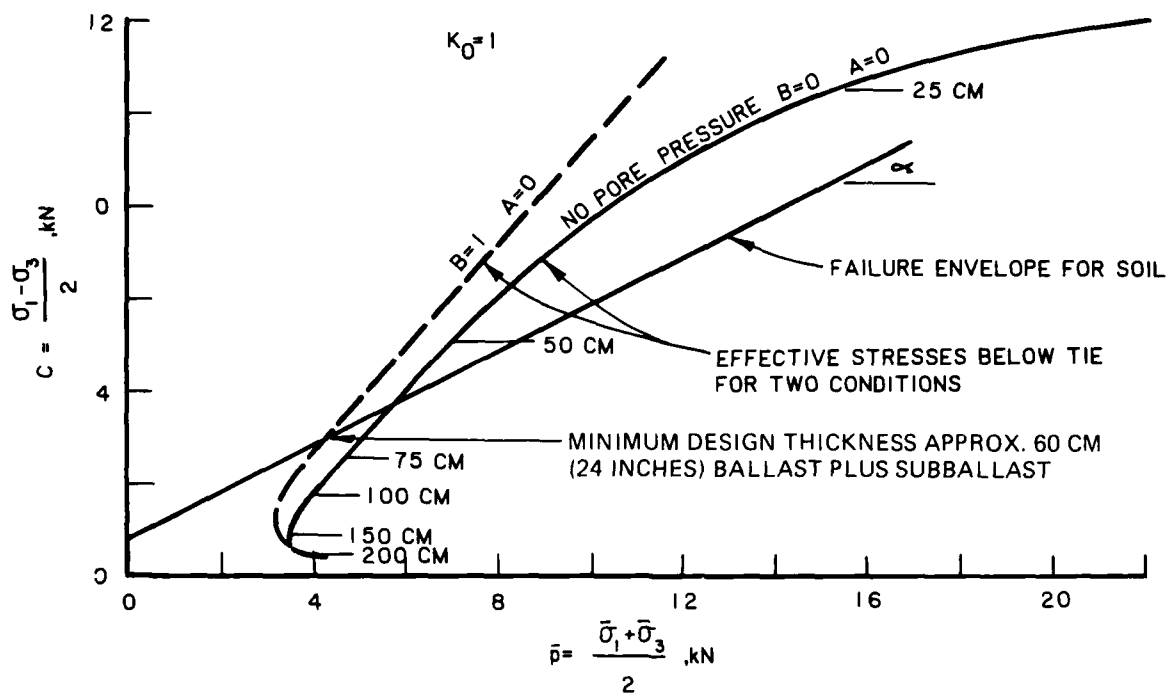
Figure 15. Indian State Railways design method (after DiPilato et al. 1983; used by permission of original authors)

Discussion

72. As seen in the previous sections, the current United States and Canadian design practice is primarily based on experience with limited use of rational analytical methods. When analytic methods are used to determine ballast and subballast thickness, the allowable pressures for the ballast and subgrade materials are often arbitrary selections rather than selections based on properties of the subgrade soils. Where North American railroads have developed standard sections, these sections are often not related to subgrade type, strength, or deformation properties.

73. Review of international design procedures indicates that the railroads reviewed all use rational, analytic methods combined with experience to determine the required ballast and subballast thicknesses. The design thicknesses are modified based on experience to take into account environmental conditions such as freeze-thaw, swelling soils, and excessive moisture. All of

the design methods reviewed have the fundamental design criterion of limiting
the subgrade stresses to an allowable level that the subgrade can support
without excessive permanent deformation. The Boussinesq elastic stress dis-
tribution for a semi-infinite half-space is widely used for estimating the
stresses below the ties. It is generally felt that the magnitude of stresses
in the track structure do not necessitate a more rigorous solution. In a
notable deviation from American design practice, the foreign railroad design
practices reviewed emphasized the importance of classifying the subgrade soils
and determining the subgrade strength and deformation properties using field
and laboratory testing.

74. The standard ballast sections required by the foreign railroads
range from 10 to 20 in. with required subballast thicknesses ranging from 2 to
22 in. The total ballast/subballast thickness required by the reviewed
foreign railroads varies from 16 to 34 in. for a 22,000-lb wheel load. In
comparison, typical North American railroads carrying 33,000- to 39,400-lb
wheel loads require 6- to 12-in. minimum ballast thickness and 0- to 12-in.
subballast thickness resulting in total minimum thicknesses up to 24 in.

PART IV: ANALYTICAL TRACK RESPONSE MODELS

Overview

75. Development of the railroad track structure into its present form has been mostly a trial-and-error process; however, over the last century railroad engineers have taken an interest in the analysis of track and its components. Early work by Winkler in determining rail stresses in longitudinal-tie track was performed considering the rails as continuously supported beams on an elastic foundation. Zimmerman, among others, used Winkler's theory for analysis of longitudinal-tie track as well as for analysis of crosstie track. Development of the analysis for crosstie track was more involved as reported by Kerr. The rail was first considered as a beam resting on discrete rigid supports, then as a beam resting on discrete elastic supports, and finally as a continuously supported beam. The continuously supported beam on an elastic foundation as presented for longitudinal-tie track ultimately prevailed for rail stress analysis of crosstie track. This approach was subsequently used by the Talbot Special Committee on Stresses in Railroad Track, which introduced the term "track foundation modulus," and is the basis for most current track design procedures.

76. In the past 20 years several researchers have developed analytic procedures to model the complex response of the track structure to load. These analytic procedures include the modified beam on an elastic foundation developed by Meacham et al., the finite beam on an elastic foundation studied by Hetenyi and modified by Barden and Harrison, and the general Boussinesq approach as presented by Ireland. Each of these analytic methods are briefly described by Selig et al. (1979). With the advent of high-speed computers and new analytical tools such as the finite element method (FEM), computerized analytic procedures have been developed that can model the response of the various track components (rails, tie plates, ties, ballast/subballast, and subgrade) to various loading conditions. These computerized analytic procedures include the Prismatic Solid Analysis (PSA) model, the ILLITRACK model, the Multilayer Track Analysis (MULTA) model, the GEOTRACK model, the Analysis of Rail Track Structures (ARTS) model, the Finite Element Analysis of Railway Asphalt Track (FEARAT) model, the KENTRACK model, the LATRACK model, and the SAFRAL model.

With the exception of LATRACK, which analyzes both vertical and lateral response, and SAFRAL, which analyzes rail bending and stress, all of the above models analyze the structural response of the track system to vertical loads. A brief description of these track response models is presented in the following sections.

Descriptions of Track Response Models

Beam on elastic foundation

77. The beam on elastic foundation method for analysis of railroad track structure has been detailed in paragraphs 35 and 36. This method is the basis for most current track design and for many track analysis procedures. However, this method has several limitations when used for analysis of track response. The primary limitation to this approach is that it does not adequately model the stress-strain behavior of the ballast and subgrade. Another limitation is the lumping of the tie-ballast-subgrade strength into one parameter, i.e., track modulus, which is difficult to determine and may not be truly representative once measured.

Prismatic Solid Analysis (PSA) model

78. The PSA model is a three-dimensional FEM model developed by the AAR. This model considers the ballast and subgrade layers separately from the rail-tie structure for developing stress and displacement influence coefficients and then imposes the compatibility of the stresses and displacements between the bottom of the rail-tie system and the top of the ballast-subgrade system to determine the solution for stresses, strains, and displacements. Foundation stress and displacement influence coefficients are calculated based on analysis of periodically loaded prismatic solids. Ballast and subgrade materials are considered to be linear elastic; however, different elements in the vertical plane may have different elastic properties. Loads must be input as periodic in the longitudinal direction. PSA is described in detail by Selig et al. in "A Theory for Track Maintenance Life Prediction" (1979).

ILLITRACK model

79. The ILLITRACK model was developed at the University of Illinois (Robnett et al. 1976) as a pseudo three-dimensional model. ILLITRACK is essentially two two-dimensional finite element models, one longitudinal, the other transverse, which when combined produce the same results as a

52

three-dimensional model at less computer cost. The output from the longitudinal analysis is used as input to the transverse analysis requiring two runs for the solution of one problem. In the longitudinal analysis the model considers point loads corresponding to wheel loads acting on a single rail supported by the tie-ballast-subgrade system. The rails and ties are represented in the model as a continuous beam (rail) supported on springs (ties). The ballast, subballast, and subgrade are represented by rectangular planer elements the thickness of which increases with depth using a pseudo-plane strain technique. This technique accounts for load spreading perpendicular to the plane allowing simulation of a three-dimensional load spread with a two-dimensional model. Use of this pseudo-plane strain technique requires input of a thickness for the plane strain section or an "effective tie bearing length," L, and an "angle of distribution," ϕ, which determines the rate of increase of the element thicknesses with depth. Tayabji and Thompson (1976) recommend an effective bearing length equal to 18 in. and an angle ϕ equal to 10 deg for analysis of conventional track structures. The transverse analysis is performed using an approach similar to the longitudinal analysis; however, the entire tie width is input as the plane strain section thickness. Nonlinear material properties are accounted for by using the resilent modulus of the ballast, subballast, and subgrade materials as an input into the model. This resilient modulus of soil materials may be determined from laboratory repeated load triaxial tests simulating the expected load levels, durations, and frequencies. Ballast and subgrade failure criteria have been incorporated into the ILLITRACK model. The principal stresses are modified at the end of each step in the solution so that they do not exceed the strength of the material as defined by the Mohr-Coulomb envelope. Raad and Thompson in a discussion attached to the article "Study of Analytical Models for Track Support Systems" (Adegoke, Chang, and Selig 1979) present a detailed description of this failure criterion.

Multilayer Track Analysis (MULTA) model

80. The MULTA model is a combination of two computer codes, namely BURMISTER and LOADS AND COMBINATIONS (LAC). The BURMISTER code uses Burmister's multilayer elastic theory to represent the ballast and soil layers. The tie-bearing area is divided into approximately equal segments and the area of each segment is converted to a circular area of uniform pressure. These uniformly loaded circular areas are used to generate stress and displacement

53

influence coefficients for the multilayer linear-elastic model. LAC is a
matrix structural analysis model that solves for the tie-ballast reactions
using the method of consistent deformations. In LAC, wheel loads are applied
on opposite rails representing axle loads while each rail is assumed to be a
finite beam supported by 11 ties. Ties are also represented as beams with one
support for each tie-ballast contact area segment. The tie support force, the
rail-tie reaction, and rail-tie displacement are all unknowns resulting in an
indeterminate structures problem. Compatibility and equilibrium equations are
used to form a set of simultaneous equations which may be solved for all of
the unknowns. The magnitude of the tie-ballast pressures determined for each
tie segment is superimposed on the roadbed system for all ties, and displace-
ments and stresses within the ballast-subgrade system are calculated using
BURMISTER. Details on MULTA can be found in "A Theory for Track Maintenance
Life Prediction" (Selig et al. 1979).

GEOTRACK model

81. Development of the GEOTRACK model is described by Selig et al. in
"A Theory for Track Maintenance Life Prediction" (1981) and by Stewart (1981).
The GEOTRACK model is based on MULTA, with modifications to take into account
the stress-dependent nature of the roadbed materials. Other major changes
incorporated into GEOTRACK were as follows:

<blockquote>

a. The modulus of the linear elastic materials in GEOTRACK was
changed to be a function of stress state rather than a constant.

b. Repeated calculations of influence coefficients were eliminated
to reduce data input and calculation time.

c. Soil parameters were computed at more locations with GEOTRACK
than with MULTA.

d. Automatic superposition of adjacent axle loads was incorporated
into GEOTRACK.

e. GEOTRACK eliminated the need for the BURMISTER and LAC codes
in MULTA.

f. GEOTRACK reduced the need for manual computation required and
the input requirements by automatically locating where stress
and displacement data were to be output based on the track
properties (tie length, tie spacing, number of ties).

</blockquote>

GEOTRACK can analyze up to four superimposed axle loads taking into account
the nonlinear stress dependent nature of the roadbed materials in an iterative
solution scheme.

Analysis of Rail Track Structures (ARTS) model

82. ARTS is a finite element analysis model developed by Raymond Turcke, and Siv (1980) at Queens University, Ontario, Canada. It is a linear or nonlinear three-dimensional finite element program for modeling a track structure under static loads. Numerical techniques are used to account for continuous stress path dependence, nonlinearity, and the material's inability to sustain tension. Either beam elements or three-dimensional elements may be used to model the rails and ties while hexahedronal and tetrahedronal elements are used in modeling the ballast and subgrade.

Finite Element Analysis of Railway Asphalt Track (FEARAT) model

83. FEARAT, a three-stage analysis based on linear elastic theory, was developed at the University of Maryland for the analysis of full-depth asphalt trackbeds. The FEARAT model can only handle a full-depth hot-mix asphalt on a soil subgrade and involves many approximations and assumptions. A brief description of this model can be found in "Hot-Mix Asphalt for Railroad Trackbeds - Structural Analysis and Design" (Huang, Lin, and Deng 1984).

KENTRACK model

84. KENTRACK is a combined finite element-elastic layered computer model developed at the University of Kentucky for the design and analysis of railway trackbeds (Huang et al. 1984). The model applies BURMISTER's layered theory and the finite element method to determine stresses and strains in the trackbed. Although this model was developed specifically for track structures having a hot-mix asphalt underlayment between the ballast and subgrade, the model is versatile enough to be applied to the design and/or analysis of conventional ballast track or concrete slab tracks with either wooden or concrete ties. Two types of failure criteria have been included in KENTRACK. The first is the maximum vertical compressive stress or strain in a specified layer (ballast or subgrade) to control permanent deformation. The second is the maximum horizontal tensile strain in the bottom of the asphalt layer (if present) to control fatigue cracking. At the present time this asphalt fatigue criterion is based on highway design criteria, which may not be valid; however, research is ongoing to determine the proper strain criteria for asphalt layers in trackbeds. The KENTRACK model also includes damage computations based on the calculated stresses and strains and a monthly or seasonal damage concept. Details on this optional feature of KENTRACK along with a detailed description

55

of the model may be found in "KENTRACK, a Computer Program for Hot-Mix Asphalt and Conventional Ballast Railway Trackbeds" (Huang, Lin, and Deng 1984), which serves as a user's guide for the model.

LATRAK model

85. The LATRAK model is a two-dimensional finite element model of track structures consisting of a pair of rails supported by a discrete set of ties. The current version of the program was developed at Tufts University, Boston, Mass., to analyze track with structural anomalies such as missing or ineffective ties and imperfect joints. This model was designed primarily to analyze rail and tie response; therefore the ballast and subgrade are modeled as a finite beam on elastic foundation and the foundation properties are input in terms of a track modulus. For each tie or load location the LATRAK model predicts vertical, lateral, and torsional rail displacements; the resultant torques, lateral, and vertical bending moments and shear forces; tie plate loads and moments; and maximum axial bending stresses in the head and base of each rail from lateral and vertical loads. The "Applications Guide to LATRAK Analysis of Railroad Track" (Perlman and Toney 1977) contains a detailed description of this model.

TRKLOD model

86. The TRKLOD model is a nonlinear finite element program being developed by the Transportation Systems Center to investigate the rail-crosstie fastener system and conditions leading to excessive gage widening. In TRKLOD the tie/fastener system is modeled using nonlinear springs, two vertical springs, and one lateral spring connected at the base of the rail. With this model nonuniform tie conditions as well as missing ties and broken rail conditions can be simulated. The model is not complete and still must undergo field validation; however, details can be found in "User's Manual for TRKLOD" (Jeong and Coltman 1982).

SAFRAL model

87. The SAFRAL model is a closed-form solution for determining the stresses in a rail under load. The model was developed at Tufts University and predicts the axial stress, vertical bending stress, lateral bending stress, and torsional bending stress at seven locations in a rail. The foundation parameters are included in the analysis by means of a vertical foundation modulus, lateral foundation modulus, and a torsional spring constant.

Curving models

88. Several models have been developed by various organizations to determine the longitudinal, vertical, and lateral forces produced as a wheel truck proceeds through a curve. These models can be used to evaluate the possibility of rail overturning, the feasibility of curve oiling, the effect of various truck configurations, and similar curving problems.

Models Selected for Additional Evaluation

89. After reviewing available literature on the various analytical models, four of the models were chosen for further evaluation prior to selecting one model for use. The criteria used in selecting the models for further evaluation were:

a. Applicablity to Army track and loading conditions.

b. Accuracy of model when compared to field measurements as reported in the literature.

c. Compatability with existing or future structural evaluation methods.

d. Required inputs.

e. Type and amount of information output.

f. Difficulty in using model (data preparation, coding, etc.).

90. Based on these criteria, the Beam on Elastic Foundation (BOEF), GEOTRACK, ILLITRACK, and KENTRACK models were chosen for additional evaluation. Details of the evaluation of these models along with the selection of a track response model for use in a track evaluation program are discussed in Part VIII of this report.

PART V: TRACK PERFORMANCE MODELS

91.   Performance models for the various track components are generally
not available. The primary reason for the lack of development of such per-
formance models is the lack of interest by commercial railroads. Currently
the AAR is completing a rail performance model, and is developing a tie per-
formance model. Performance prediction models for the ballast and subgrade
have been proposed; however, work has not begun on these models.

92.   The AAR rail performance model predicts the optimum life of a rail
for a particular situation and gives an indication of the best time to replace
the rail based on a life-cycle-cost analysis. The model is based on the
fatigue life of the rail. Inputs include: rail properties (weight, section,
section modulus), axle loads, cost of new rail, cost of relaying rail, cost of
defect detection, cost of a derailment, and the number of defects per mile
when the rail is replaced.

93.   The tie performance model is being developed by the AAR in a manner
similar to the rail performance model; however, the model is not complete at
this time.

PART VI: STRUCTURAL EVALUATION METHODS

Overview

94. Commercial railroads have little need for a structural evaluation method in that the traffic loads and densities carried by these railroads quickly reveal structural problems that must be corrected. The light traffic and relatively low loads presently being carried over military track is insufficient to reveal any structural problems that may exist. These structural problems would become apparent only under mobilization traffic or other heavy loads and traffic. Evaluation of military trackage is therefore necessary in order to locate and correct these defects before a conflict, with its increased traffic and heavy loads, exposes the defects, possibly at a time and location which would set back the defense effort.

95. The evaluation of military railroads should be a combined functional, rail defect, and structural evaluation. Functional evaluation includes track geometry measurements and visual inspection of special track work (switches, turnouts, etc.) to ensure safe and efficient movement of trains. Rail defect testing would enable detection of internal rail defects that might result in rail breaks. Structural evaluation is the evaluation of the entire track structure to determine whether it is capable of supporting the expected loads and traffic densities without undue displacement and/or loss of servicability. Methods of structural evaluation will be discussed in the following paragraphs with rail defect testing and functional evaluation discussed in Part VII of this report.

Structural Evaluation Methods

96. Most conventional soils and pavements evaluation techniques are adaptable to evaluating railroad track structures. Over the years the railroad industry has based most of its track evaluations on the track modulus value, as determined from track deflection measurements under a loaded car, and the beam on elastic foundation theory. In the past 10 years new evaluation methods have been developed; however, they have not been widely used except as research tools. Some of the newer devices and methods developed for pavement evaluation may be applicable to track evaluation. In the following

sections various structural evaluation techniques that have a potential for use in evaluating military trackage will be discussed.

Destructive testing

97. <u>Test pits.</u> Test pits excavated into the track structure are the most direct way of determining the engineering properties of the materials comprising the track structure. Excavation of test pits also enables collection of material samples for laboratory testing. The primary objective of using test pits in evaluating the track structure is to determine the in situ strength and physical properties of each of the materials comprising the track structure, primarily the ballast and subgrade.

98. Several researchers have reported methods for determining the in situ strength of ballast and subgrade materials. In "Mechanics of Ballast Compaction," Vol 2 (Selig, Yoo, and Panuccio 1982b), the various field test methods for determining the in situ strength of ballast materials, including field density and plate-bearing tests, are reviewed. After a comprehensive review of existing density test methods, a study was undertaken to develop a new ballast density test procedure and laboratory reference test that would be more suitable to ballast density measurement. Details of this water replacement type test may be found in "Mechanics of Ballast Compaction," Vol 2 (Selig, Yoo, and Panuccio 1982b), and in the <u>ASTM Geotechnical Testing Journal</u> article "Railroad Ballast Density Measurement" (Yoo, Chen, and Selig 1978). Selig, Yoo, and Panuccio (1982b), also reviewed the various plate-bearing test procedures currently available and recommended a 5-in.-diam plate for determining the ballast bearing index. The test equipment and procedures are described by Selig, Yoo, and Panuccio (1982b) and in the <u>ASTM Geotechnical Journal</u> article "Apparatus and Procedures for a Railroad Ballast Plate Index Test" (Panuccio, Wayne, and Selig 1978).

99. The strength of the subgrade materials may be determined from conventional in-place tests such as the field CBR or the plate-bearing test. The CBR is a measure of the resistance of soils to the penetration of a standard 3-in.^2 piston; it is determined by comparing the bearing value obtained from a penetration-type shear test with a standard bearing value obtained on crushed rock (average value of tests and a large number of samples). The standard results are taken as 100 percent, and values obtained from other tests are expressed as percentages of standard. The plate-bearing test is a bearing capacity test in which a known load is placed on a nest of circular plates and

the resulting deflection is measured. The modulus of soil reaction, K , is defined as the ratio of the applied pressure in pounds per square inch to the average deflection measured at the 10-psi load increment. Conventional sand-cone, water displacement, or nuclear methods may be used to determine in situ density of the subgrade soils.

100. Laboratory testing. During excavation of test pits, samples of ballast, subballast, and subgrade should be obtained for laboratory testing to determine the physical characteristics of these materials. Selig, Yoo, and Panuccio in "Mechanics of Ballast Compaction," Vol 1 (1982a), review the various laboratory tests used for characterization of ballast materials. Conventional soils physical property tests such as sieve analysis, Atterberg limits, and moisture content determination may be used to characterize the subgrade materials. Triaxial-type tests may also be used to characterize ballast and subgrade materials. Static triaxial tests are most often used to measure the shear strength, apparent cohesion, and angle of internal friction of materials. The repeated-load triaxial test is a triaxial test in which the vertical load is cycled while the confining pressure is held constant. The resilient modulus, E_R (defined as the repeated deviator stress divided by the recoverable strain), can be determined using this type of repeated load triaxial test, and is one means of characterizing the strength of a material. Details of the resilient modulus as related to track materials may be found in: "Technical Data Bases Report-Ballast & Foundation Materials Research Program" (Robnett et al. 1975); "Development of Structural Models and Materials Evaluation Procedures - Ballast and Foundation Materials Research Program" (Robnett et al. 1976); "Mechanics of Ballast Compaction, Vol 1" (Selig, Yoo, and Panuccio 1982a); and "A Theory for Track Maintenance Life Prediction" (Selig et al. 1981). A detailed description of resilient modulus test apparatus and test procedures is presented by Barker and Brabston in "Development of a Structural Design Procedure for Flexible Airport Pavements" (1975). The report "Ballast and Subgrade Requirements Study, Railroad Track Substructure - Materials Evaluation and Stabilization Methods" (Simon, Edgers, and Errico 1983) contains an extensive summary of field and laboratory tests suitable for use in the characterization of ballast, subballast, and subgrade materials.

101. Use of test pits and laboratory testing of materials for evaluating the structural capacity of track structures has the distinct advantage of enabling the evaluator to test the materials in place with minimum disturbance

and to examine the materials removed from the track structure. Disadvantages of using test pits include:

$\underline{a}$. Traffic delays due to the necessity of closing the track during testing.

$\underline{b}$. The time and labor required to excavate a pit and perform in-place tests and laboratory testing which results in a relatively high cost per mile for evaluation.

$\underline{c}$. Disturbance of the track structure from excavation of the pit.

102. Data obtained from test pit evaluations of track materials could be used to evaluate the track structure directly if the track design were based on some parameter such as subgrade strength determined from a CBR or plate-bearing test. Even if the material properties as determined from field and laboratory tests were not directly related to a design system, correlations between field test and resilient modulus or between soil properties and the Young's modulus, E , of track materials could be used to determine the modulus value of ballast and subgrade materials for input into a track response model.

103. Small aperture testing. As an alternative to test pits, a small aperture CBR test setup could be used to determine subgrade strength. With this method a 6-in.-diam auger hole is made through the ballast and a conventional CBR test is performed through the hole to determine subgrade strength. This method is a good rapid means for determining the subgrade strength and obtaining samples of the ballast and subgrade for visual classification and moisture content determination, although the auger hole is too small for determining in-place density.

104. The advantage of small aperture testing is that direct strength measurements may be made and material samples obtained more rapidly and with less track disturbance through the auger hole than with test pits. Disadvantages are the same as with test pits, except the time required and disturbance caused is considerably less. Other disadvantages are that only a small sample of material is obtained and only one CBR test may be made per hole, requiring at least three auger holes per site to determine a representative subgrade strength value.

Cone penetrometer testing

105. The cone penetrometer may be used to provide information on the type and strength of track materials. In the most widely used cone penetrometer test (CPT) a penetrometer with a 10-cm^2 base area and 60-deg apex angle

is advanced vertically into the soil at a constant rate of 2 cm/sec. A friction jacket advances simultaneously (electrical cones) or alternately (mechanical cones) with the tip. The force necessary to maintain this constant rate of push (tip resistance) and the side friction is measured and recorded. The continuous nature of the CPT allows thin layers of material or small areas of nonuniformity to be detected that might otherwise be missed. Details of CPT procedures can be found in "The Measurement of Soil Properties In-Situ" (Mitchell, Guzikowski, and Villet 1978) and Cone Penetration Testing and Experience (Norris and Holtz 1981).

106. A significant amount of work has been conducted in correlating the CPT to the Unified Soil Classification System (USCS). Douglas and Olsen (1981) conclude that the CPT is ideal for site investigation and profiling because it is repeatable, usually correlates well with the USCS, and provides a clearer overall picture of in situ conditions than other exploration methods. Bukoski and Selig (1981) report good results using the CPT to characterize the in situ properties of railroad subgrade soils. Work performed at WES (Ledbetter in press) using the CPT for evaluating track structures indicates that cone penetration testing yields very good relative strength comparisons between material layers. Data obtained from the CPT would be compatible with other test methods and applicable to use with the various track response models.

107. Advantages of cone penetration testing are the relatively short time required per test and the ability to detect small layers of strength variability in the track structure. The primary disadvantage is that direct inspection of the track materials is impossible.

Standard penetration test

108. The Standard Penetration Test (SPT) is a field test used to obtain disturbed samples of the substrata and provide information regarding the dynamic penetration resistance of the tested materials. In the SPT, a standard split-spoon sampler is driven 18 in. into the soil with blows from a 140-lb weight falling 30 in. Samples thus obtained are disturbed primarily due to the large area ratio of the sampler. The samples are used to provide information regarding the stratification of the soils and for simple laboratory tests which do not require undisturbed samples. Penetration resistance measured in blows per foot is an index which has been used for correlations to strength, density, and compressibility in particular types of soil. The

63

measure of resistance is obtained by counting the blows needed to drive the split-spoon sampler for three consecutive 6-in. increments. Summing the blows from the last two increments gives a number termed the SPT N value. Testing equipment and procedures can significantly affect the results of this test. The primary advantage of the SPT is the collection of soil samples for laboratory testing. Disadvantages include difficulty in getting repeatable results due to the operator-dependent nature of the test and the large variability often obtained in the test results (Mitchell, Guzikowski, and Villet 1978).

Track modulus testing

109. The track modulus value has been described as the cornerstone of the beam on elastic foundation theory (Zarembski and Choros 1979). The track modulus, U , was first proposed by Winkler as a fundamental parameter related to both the applied load and resulting track deflection. The track modulus attempts to quantify in a single term the combined effects of ties, ballast, and subgrade. The rail stiffness, while entering directly into the beam on elastic theory, is not included in the track modulus term. Details of the development and use of the track modulus are described in "On the Measurement and Calculation of Vertical Track Modulus" (Zarembski and Choros 1979) and in the first and second progress reports of the Special Committee on Stresses in Railroad Track (American Railway Engineering Association 1980).

110. In the years following the Talbot Committee's work on measuring track modulus four methods of calculating the track modulus from field measurements have been used. The first, sometimes known as the Deflection Curve Method, is the method used by the Talbot Committee and involves measuring the rail deflection at several locations along the entire length of track depressed by a single-wheel load. This method assumes that the track modulus is proportional to the applied load divided by the total area under the deflection curve for the track section. The major advantage of this method is the averaging effect of using a large area which compensates for track discontinuities that may be present. Three major disadvantages of using this method are (a) the large number of deflection measurements required in order to accurately determine the shape of the deflection curve, (b) no accounting for slack in the track structure, and (c) no accounting for differing rail sizes. To try to account for slack in the track structure later researchers devised a second method using the difference in deflections measured under a light car and heavy car. The track modulus is then computed as the difference between

64

the light and heavy load divided by the net area between the deflection curves. While this eliminates the effect of free play, the number of required deflections is doubled. The third method for determining track modulus from field test data uses a modified version of the beam on elastic foundation theory. This method uses the Winkler equation (Equation 1) to calculate the track modulus and has the advantages of (a) requiring only one deflection measurement point and (b) taking rail stiffness into account resulting in an averaging effect over the length of the depressed track section. This method is recommended by Hay in "Railroad Engineering" (1982) for use in calculating the track modulus. Kerr (1982) points out that a major shortcoming of using this third method of track modulus computation is the requirement for a single-axle load to make the necessary deflection measurements. In "A Method for Determining the Track Modulus Using a Locomotive or Car on Multi-Axle Trucks" Kerr (1982) presents the development of a fourth method for track modulus determination. Kerr's method is very similar to the third method described previously; however, any type of rolling stock can be used to obtain the deflection measurements. This method of determining track modulus has been used with good results on one Army installation as reported by Amons (1983).

111. The track modulus value for a given track will vary with the load used to determine the deflection readings. Because of this, most researchers agree that the track modulus should be determined at a load level corresponding to the traffic loading experienced by the track.

112. The track modulus value is a required input when using the Beam on Elastic Foundation Theory. The other track response models selected for additional consideration do not require the input of a track modulus value, essentially limiting its use to the Beam on Elastic Foundation Theory. Another disadvantage is that no information on material type or strength is obtained since no material inspection or in-place soils test are performed. Advantages are the nondestructive nature of the test and that any loading vehicle may be used to determine the track modulus.

AAR Decarotor Track Strength Test Car

113. The Decarotor Track Strength Test Car was developed by the AAR as a part of the Track Strength Characterization Program. The Decarotor is a specially designed test system capable of independently applying simultaneous lateral and vertical loads to each rail while the test car is in motion. This

65

system uses a pair of instrumented loading axles controlled through a closed loop servo-hydraulic system to maintain a constant load level permitting the direct measurement of lateral rail deflections under a uniform load. Vertical load and lateral load along with unloaded and loaded gage measurements are made. The gage measurements at the constant load provide a measure of the gage restraint capacity of the track. The Decarotor must be connected with a data recording car for operation and requires a five-person crew plus a loco-motive and crew to provide mobility. Average testing speed is 5 mph with a maximum test speed of approximately 7 mph. A description of the Decarotor Track Strength Test Car can be found in "Preliminary Field Evaluation of a Track Strength Test Vehicle" (Zarembski, McConnell, and Lovelace 1980).

114. The Decarotor car is designed to measure the lateral strength of the track structure and does not have the capability to measure vertical de-flection under load. Because of this, the Decarotor results cannot, at this time, be used as input into any of the vertical track response models. Other disadvantages are the requirement for a four-man crew plus a train crew for mobility and the fact that no material inspection or material strength data are obtained from this test method. Advantages include the nondestructive nature of the test and the continuous measurements obtained.

ENSCO track stiffness measurement system

115. The ENSCO, Inc., track stiffness measurement system is a dual profile measurement system developed to measure track stiffness. Track stiff-ness data are obtained from measurements of track profile obtained from standard midcord-offset (MCO) and profilometer techniques using a track geometry measurement car operating at normal speeds. The dual profile system measures the response of track to a distributed load. When MCO is measured by a beam system attached to a three-axle truck, the load is stationary with re-spect to the three measurement points. When the inertial profilometer output is converted to MCO, the relative deflections with the load in three different positions are computed. The resulting profilometer MCO is smaller than the beam MCO, and the difference between the two readings results in a deflection associated with the load only. These deflection values are a measure of the track deformation under a railcar truck and can be directly related to track stiffness, track compliance, or track modulus. Details of the ENSCO track stiffness measurement system can be found in "Track Stiffness Measurement System Evaluation Program" by Hayes, Joshi, and Sullivan (1979). The track

modulus data obtained from this system could be used in conjunction with the beam on elastic foundation analysis or the deflections, track stiffness, and track compliance could possibly be related to some other type of evaluation method.

116. Advantages of this system are the nondestructive nature of the test, the continuous strength profile obtained, and the mobility of the system because it will attach to any type of vehicle. Disadvantages are the fact that this equipment was specially developed and is not commercially available, that the system may require a three- or four-person crew, and that no material inspection or soils tests are obtained.

Geophysical methods

117. Geophysical methods of determining the elastic constants of track materials have previously been used in evaluating track structures. Both vibroseismic and impedance test methods have been used (Cooper 1975).

118. _Vibroseismic tests._ The vibroseismic test method is based on measuring the propagation of surface waves generated by a controlled vibratory source operating at discrete frequencies. Determining the wave velocities over a range of frequencies provides a means of deriving the elastic constants of the track substructure materials as well as their variation with depth. Both vibratory and surface refraction seismic investigations are used in the vibroseismic test method.

119. In the vibratory investigation the frequency is controlled and wave length is the measured variable. The length of the surface Rayleigh waves is actually measured and the shear waves are considered to propagate at the same velocity. The shear modulus, G , can be computed from the shear wave velocity, allowing Young's modulus, E , to be computed from the shear modulus and Poisson's ratio.

120. In the refraction seismic phase of the investigation data are obtained with a commercially available portable seismic unit. The compression wave travel time from the source to the geophones, located at various distances away from the source, obtained from the test is plotted and the slope of the line through the points determined. The inverse slope of this line gives the compression wave velocity, V_c , of the material. A change in slope of this line indicates that the wave has passed through an interface between two layers having different velocities, allowing the interface depth to be calculated. Both forward and reverse refraction profiles are obtained so that

67

the velocity variations can be corrected to obtain the true velocity of each layer. The true velocity and interface depths obtained from the seismic data can be used to determine the substructure velocity profile.

121. The Young's modulus data obtained from the vibratory investigation along with the velocity depth profile obtained from the seismic investigation can be used as inputs into the various analytical models to determine track response. Advantages are the nondestructive nature of the test and the structure profile and material property data received from the test. Disadvantages are the time required to run the test, the complicated data analysis required, and the need for a large vibrator.

122. Impedance tests. Mechanical impedance can be defined as a quantitative measure of structural response to a known vibratory force. A vibrator is used to excite the system being tested with a sinusoidal force and the structural response is measured with velocity transducers located at the points of interest. Details of the impedance test method and the complex calculations involved in analyzing the data are presented in "Mechanical Impedance Evaluations of the Kansas Test Track: Pretraffic and Posttraffic Tests" (Cooper 1979). After the impedance data have been acquired, dynamic properties of the system such as stiffness, damping, participating mass, and resonant frequency can be interpreted from impedance plots.

123. The impedance method can be used to evaluate the effects of system variations; however, the results from this type of evaluation are not readily adaptable for use in already existing track response models. Other disadvantages are the same as those listed in paragraph 121 for the vibroseismic test method.

Load-deflection testing to
determine material elastic properties

124. The load-deflection response of a track structure is similar to that of a highway or airfield pavement, therefore nondestructive pavement testing techniques should be adaptable to rail track structures. The deflection of the track structure under a known load can be measured and either the elastic layer theory or the finite element method can be used to determine Young's modulus, E , values for the various structural layers. The WES 16-kip vibrator, which has been used in previous railroad evaluation work (Ledbetter in press), and the falling weight deflectometer (FWD) are two devices which have potential as track evaluation tools. The data obtained from

68

this type of evaluation would be compatible with either of the four track response models chosen for additional evaluation.

125. Advantages of this test method include the nondestructive nature of the test and the determination of material properties from the test results. Disadvantages are that the structure (material types and thicknesses) must be known and the need for a large load input.

Ground penetrating radar

126. Although not a structural evaluation tool in the sense of a deflection measuring device, ground penetrating radar has potential for use in evaluation of track structures. Previous research has demonstrated the feasibility of using radar for delineating the material layer interfaces and detecting areas of high moisture content in a track structure (Lundien 1979). Work conducted by the AAR demonstrated that radar measurements were repeatable over the same trackage and that radar could be used to locate the ballast-subgrade interface (So, Hutcheson, and Breese 1980). Radar has the capability of rapidly profiling a track structure, minimizing the number of soil borings required, and giving a general strength profile of the track structure. Recent advances in the state of the art of ground penetrating radar for pavement profiling is applicable to railroad track structures and carries some potential as an evaluation tool.

127. The primary advantage of using ground penetrating radar is the continuous determination of the material layer depths. The primary disadvantages are the extensive data analysis required and the fact that soil properties cannot be obtained from the radar data.

PART VII: RAIL DEFECT TESTING AND FUNCTIONAL EVALUATION METHODS

Overview

128. As stated previously, the evaluation of a track structure should
be a combined structural-functional evaluation. Although commercial railroads
do not consider structural evaluation necessary, they make regular use of rail
defect testing and geometric deviation testing. The following paragraphs
describe current practice in these areas and briefly describe the equipment
and procedures used for the functional evaluation of track. New methods and
tools currently being developed that have potential for use in functional
evaluation of military trackage are also discussed.

Rail Defect Testing

129. Visual inspection can locate obvious external rail defects, but
cannot detect internal defects, such as transverse fissures, which cannot be
seen until the rail actually breaks. The purpose of rail defect testing is to
locate rail defects that are potential problems so these defects can be re-
moved before they cause a rail break and possible derailment.

Detection methods

130. The two most widely used automated methods for rail flaw detection
are induction and ultrasonic detection. In the induction method an electrical
current is passed through the rail by a series of brushes riding on the rail
creating a magnetic field around the rail. Search coils riding in a carriage
over the rail detect any change or distortion in the field and send an ampli-
fied electrical current to the data recording equipment where a "blip" on a
paper tape indicates a defect. At the same time the defect is recorded on the
tape, a blob of paint is dropped on the rail to mark the defect location.
When a defect is detected but cannot be visually verified, a hand test is
made. This hand test consists of sending a heavy current through the area
surrounding the suspected defect and then measuring the drop in potential
across a 1/2-in. gap. The exact location of an internal defect can be deter-
mined because above a fissure the potential drop increases rapidly to a maxi-
mum as the reduction in rail cross section caused by the fissure increases
electrical resistance. Ultrasonic test systems have an advantage over

70

induction-type testing in that they can test through rail joints and detect rail defects that would be missed with induction testing. With induction testing the equipment falsely indicates a defect when it crosses over joint bars, bolt holes, and bolts. Therefore, a section adjacent to the joint is often cut out of the test pattern. In ultrasonic rail defect testing quartz-crystal transducers direct ultra high frequency sound waves into the rail. The sound waves are reflected back to a receiver with a variation in signal strength and pitch that depends on the rail cross section, enabling the detection of defects. The detection area varies with the angle at which the sound is introduced, and several different input angles must be combined in order to check the rail for both head transverse defects and web and base defects. Details of the various types of rail defect testing procedures can be found in "Railroad Engineering" (Hay 1982).

Types of test vehicles

131. There are three basic types of rail defect testing equipment currently in use. These are full-size, track-bound test cars; high-rail test vehicles; and portable ultrasonic detection equipment. The full-size track-bound test cars are self-propelled and generally contain both induction and ultrasonic type detection equipment. These cars are capable of detecting all types of rail defects at speeds up to 13 mph and averaging over 40 miles per day. High-rail test vehicles generally contain only ultrasonic detection equipment and are designed for testing industrial type trackage. High-rail vehicles have the advantage over full size cars of increased mobility because they are not confined to the track. Portable ultrasonic units, either hand-held or mounted on a small cart, are available for spot testing or for use in areas with limited access.

Use of rail flaw detection

132. Most commercial railroads have a regularly scheduled program of rail flaw detection. In 1982 rail flaw detection was used to identify rail defects at 24 Army installations in the United States. This work was conducted using a full-size test car, a high-rail test vehicle, and a hand unit depending on the track location and conditions. Communication with Sperry Rail Service Division, the leading provider of rail flaw testing services in the United States, indicated that for typical CONUS Army tracks their normal test procedure would be to use an all-ultrasonic high-rail vehicle. The cost for this unit is about $200 per hour for a minimum 8-hr day. Depending on

71

defect density, rail conditions, and operating conditions, 8 to 15 miles of
track can be tested per day.

Track Geometry Inspection

133. If a track structure is adequately strong to support the loads it
is carrying and the rails are defect free but the track geometry is not within
required limits, the operation of trains over the track will be limited.
Track geometry involves:

a. Cross level. The two rails must be at the same elevation on
tangent track. Poor cross level accentuates rocking and
can lead to derailments.

b. Superelevation. A constant elevation of the outside rail over
the inner rail must be maintained on curves as well as uniform
rate of change on spirals.

c. Profile. As cross level relates to transverse track elevation,
profile relates to elevation along the longitudinal axis, that
is, adherence to the established grade without dips and sags.

d. Warp. Warp relates to the cross level measured diagonally
from one corner of a car to the other. A low spot under the
left front wheel and another low spot under the right rear
wheel would represent an unfavorably warped situation.

e. Gage. The inside distance between the gage corners of the
rails, measured 5/8 in. below the top of rail, must be within
the limits established for safe and proper gage. Excess gage
widening or play increases lateral movements, hunting, and
nosing; if the gage is too wide, the car wheels may drop off
the rails.

f. Curvature. Uniformity in the degree of curve and smoothness
in spiral transitions are important elements in maintaining
proper alignment.

The purpose of track geometry inspection is to ensure that the geometric re-
quirements set forth in the "US Army Rail Maintenance Standards" (Head-
quarters, Department of the Army in press) are met in order to facilitate the
safe and efficient movement of trains. Commercial railroads currently use
methods of track geometry inspection including: on-the-ground inspection
(track walker), full-size track bound geometry cars, and high-rail geometry
vehicles. A new type of geometry measurement called the Cross-Level Index
(CLI) is being developed by the Department of Transportation's Transportation
Systems Center to evaluate track geometry in relation to the potential for car
rollover.

Inspection methods

134. <u>On-the-ground inspection (track walker)</u>. On-the-ground inspection made by a person either walking or on a motor car with frequent stops is a primary means of performing track inspections. Using various tools such as a track level, track gage, pocket rule, taper gage, straightedge, and string-line, the inspector checks the track geometry and rail wear at various locations along the track. During these inspections, indications of rail defects should be observed. On-the-ground inspections of turnouts, rail crossings, and highway crossings are mandatory even when automated inspection techniques are used. On-the-ground inspection has the advantage of allowing the inspector to observe details of the track that might not be discernible from a locomotive, high-rail vehicle, or full-size test car. The primary disadvantage of this type of inspection is that the geometry measurements are not made under load, resulting in the true geometric conditions not being measured. A second disadvantage is the lack of comparability between inspectors, making it difficult to make decisions regarding allocation of maintenance money between installations.

135. <u>Full-size geometry test car</u>. Track geometry inspection has been automated through the use of track geometry test cars. The FRA has developed a fleet of test cars designed to measure track geometry under load. Many commercial railroads have also constructed and are using track geometry cars. Hay in "Railroad Engineering" (1982) presents details of the FRA test car operations including descriptions of how the profile, vertical displacement, gage, cross level, and track curvature are made. The principal use of the FRA test cars is to monitor adherence to individual railroad and FRA safety standards. Using data obtained by the test car, areas of track that are unacceptable when compared to the FRA track safety standards can be identified, along with areas which may be acceptable from a safety standpoint but unacceptable from a maintenance standpoint.

136. The full-size FRA test cars are approximately 85 ft long, weigh 55 tons, and have 13.75-ton axle loads. All of the test cars except one, which is self-propelled, are designed to be pulled by a locomotive or attached to a regularly scheduled train. These test cars were designed for high-speed testing with operating speeds between 15 and 150 mph. The results obtained with these test cars lose accuracy when data are collected at speeds below 15 mph. The minimum required crew size is four, plus a locomotive and crew to provide

73

mobility for the test car. Communication with ENSCO Inc., the FRA's prime contractor for test car operation, indicated that operating costs are in the range of $3,500 per day. Annual maintenance and upkeep costs are in the range of $70,000 to $90,000. The cost of transporting a full-size test car over a commercial railroad varies from $500 to $2,000 per day depending on the railroad company, distance traveled, and other factors.

137. <u>High-rail track geometry vehicles.</u> The principles of track geometry measurement developed for use with the full-size, track-bound test cars have been adopted for use in high-rail vehicles. Using high-rail vehicles has the advantages of increasing the mobility of the geometry inspection equipment and reducing the cost per mile of testing. The principal disadvantage of using a high-rail geometry vehicle is that the load on the track produced by the high-rail vehicle's test wheels is not representative of the loads the track experiences under train traffic.

138. ENSCO, Inc., operates two high-rail test vehicles for the FRA; however, they feel that the results obtained with the high-rail geometry vehicle are not as detailed nor as accurate as the results obtained from a full-size test car. These FRA owned high-rail vehicles are equipped with ultrasonic rail flaw detection equipment as well as geometry inspection equipment; however, the normal procedure is to perform geometry inspection and rail flaw detection separately. The cost for geometry testing with this equipment is about $25 to $30 per mile with annual maintenance and upkeep costs in the range of $25,000 to $30,000. The normal crew size for the high-rail geometry vehicle is two, and the data obtained are the same as obtained on a full-size test car. Communication with Plasser American Corporation, makers of a high-rail track geometry vehicle, indicates that their high-rail track geometry vehicle is capable of testing at speeds up to 25 mph, although this is a safety limitation related to the use of the high-rail wheels and not an equipment limitation. Details of this Plasser American Corporation track geometry vehicle can be found in the <u>Railway Track and Structures</u> article "Truck-Mounted Track Geometry Car Has Big-Car Capabilities" (Nov 1979) or in the company literature on the vehicle. Plasser American also provides geometry inspection services using this type of vehicle. The current (1984) rate charged by this company for track measuring and analysis varies from $60 per mile for less than 100 miles tested to $38 per mile for 1,000 miles or more of track tested.

74

139.   The Iowa Department of Transportation (IDOT) has performed an extensive study on the use of high-rail track geometry vehicles with the results presented in "Track Geometry Measurement by High-Rail Vehicles" (Sherfy 1979). One of the aspects of this study was the comparison of high-rail geometry vehicle output with output from a full-size FRA test car taken at the same location.   It was found that track geometry measurements obtained by a high-rail vehicle could be modeled to match measurements obtained by a full-size test car.   IDOT is currently using a high-rail vehicle for geometry inspections on commercial railroads as a supplement to normal state inspections.

Other means of geometry testing

140.   Bolt-on geometry package.   A prototype geometry testing package has been developed by ENSCO, Inc., for the FRA that has potential for use on Army trackage.   This package is a geometry measurement system that is completely portable and can be mounted on any railroad car in approximately one day.   This system gives complete track geometry data and has the advantage of providing geometry measurements obtained with the track loaded by the car most critical to the track.   The major disadvantage of this system is lack of real-time data processing and the time lag involved in data processing and analysis upon completion of the field testing.

141.   Cross level index measurement.   The cross level index measurement system was developed at the Department of Transportation's Transportation Systems Center (TSC) to measure the relative change in track cross-level.   The cross-level measurement device is a completely portable system consisting of a gyroscope which is mounted on a locomotive axle and a microprocessor for collecting, recording, and outputting the data.   The gyroscope is controlled by the microprocessor and measures the relative elevation change between the rails at 3-ft intervals along the track.   The collected data are saved on a cassette tape while also being output on a paper strip chart.   Also calculated and output is the CLI value.   The CLI is the root-mean-square (RMS) of the cross-level measurements taken at approximately 3-ft intervals on the previous 400 ft of track.   A 0.3 CLI value has been selected (empirically but apparently somewhat arbitrarily) as a limiting value above which car overturning is possible if the section is traversed at the critical speed of 15 to 18 mph. The 0.3 limiting value was determined for a loaded 100-ton hopper car having a high center of gravity which is very susceptible to overturning.   Each type of railcar will have a different roll angle; therefore, the 0.3 limiting CLI may

75

not be valid for all types of cars. The cross-level measurement device is operational at any speed with testing performed at the normal track operating speed. The CLI is a possible tool for maintenance management; however, the 0.3 CLI limiting criterion may not be valid for Army rolling stock. The primary problem with using the CLI is its acceptance, in that it is not related directly to the FRA Track Safety Standards, nor is it yet recognized as a safety parameter. Another disadvantage of the CLI device is that it can only be mounted on certain types of locomotives (the GP-9 and GP-40) and would have to be modified for use on Army equipment. Additional development of this system will eliminate this disadvantage in the near future.

PART VIII: SELECTION OF TRACK RESPONSE MODEL

142.   As outlined in paragraphs 89 and 90 of this report, four track response models were chosen for additional evaluation prior to selection of a model for use in predicting track response. The models chosen were: Beam on Elastic Foundation (BOEF), GEOTRACK, ILLITRACK, and KENTRACK. Descriptions of these models are presented in Part IV of this report.

Criteria for Selection

143.   The criteria for selection of a track response model to be used in the analysis of military railroads were:

a.   Required inputs:

(1) Type of input data required.

(2) Amount of input data required.

(3) Source of input data (such as laboratory tests or field tests).

b.   Data preparation and input:

(1) Amount of data preparation prior to input.

(2) Time required for data preparation.

(3) Time required for data input.

c.   Computer program run time:

(1) Central processing unit (CPU) time.

(2) Cost.

d.   Output:

(1) Type of information output.

(2) Amount of output.

(3) Accuracy of output when compared to field data.

e.   Program documentation:

(1) Is program documentation adequate?

(2) Is program documentation current?

f.   Compatability with existing or expected future structural evaluation methods.
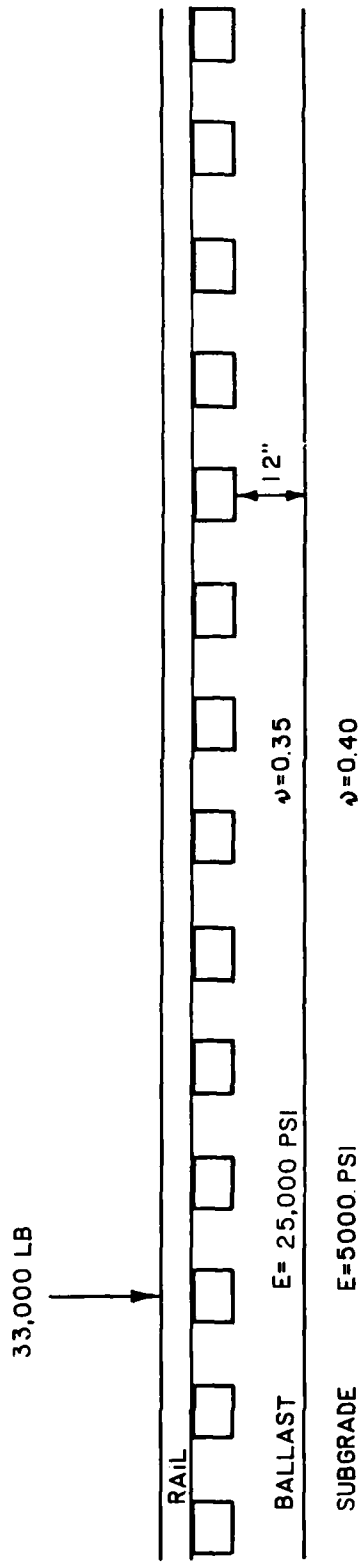
77

Evaluation of Candidate Models

Test cases

144. In order to compare and evaluate the candidate models, three test cases, designated cases 1 through 3, were worked using each of the models. Case 1 is a single-axle load (33,000-lb wheel load) on linear ballast and subgrade, case 2 is a two-axle truck (33,000-lb wheel loads at 70 in. center-to-center) on a linear ballast and subgrade, and case 3 is two adjacent two-axle trucks on a linear ballast and subgrade. Figures 16 through 18 present the track structure properties and loading conditions for each case.

145. In a previous research study the University of Kentucky Department of Civil Engineering had obtained the computer codes for several track response models, including the GEOTRACK and ILLITRACK programs for comparison with the KENTRACK program. Because of the availability of each of the candidate models on the University of Kentucky IBM 3081 computer system, a visit to Lexington was made and the test cases were run on the computer system. Each of the test cases was also worked using the BOEF analysis and the manual methods presented in Part III of this report.

Comparison of models

146. Required inputs. Table 7 presents a list of the data required for input into each of the candidate models. The inputs for the three computer models are similar; however, some variations in the input occur between models, depending on the options selected. Both ILLITRACK and KENTRACK consider tie-ballast separation automatically, while in GEOTRACK the user has the option of specifying allowance for this condition. ILLITRACK requires that the type of analysis (whether longitudinal or transverse) be input. In order to make a complete ILLITRACK run a longitudinal analysis is performed, followed by a transverse analysis using input data obtained from the longitudinal analysis. The GEOTRACK and KENTRACK programs are one-step models which perform all computations in one step, eliminating the need for a two-stage analysis. Two other inputs required only in ILLITRACK are the effective tie-bearing length, L , and the angle of distribution, ϕ . These values are required inputs in the pseudo-plane strain analysis finite element method that is used in ILLITRACK. In order to calculate the ballast pressure both GEOTRACK and KENTRACK divide the tie length into segments requiring the number of segments be input. Only KENTRACK allows the tie cross section to be varied along the

78

RAIL

BALLAST   E= 25,000 PSI   ν=0.35

SUBGRADE   E=5000 PSI   ν=0.40

33,000 LB

12"

MATERIAL PROPERTIES:

RAIL:
SECTION	:	90 RE-A
AREA	:	8.82 IN.^{2}
SECTION MODULUS (BASE)	:	15.23 IN.^{3}
MOMENT OF INERTIA	:	38.70 IN.^{4}
YOUNGS MODULUS	:	30x10^6 PSI

TIE:
WIDTH	:	9.0 IN.
DEPTH	:	7.0 IN.
LENGTH	:	102.0 IN.
MOMENT OF INERTIA	:	257.25 IN.^{4}
YOUNGS MODULUS	:	1.5 x10^6 PSI
CENTER-TO-CENTER SPACING	:	20.0 IN.

BALLAST:
YOUNGS MODULUS	:	25,000 PSI
POISSON'S RATIO	:	0.35
UNIT WEIGHT	:	0.064 PSI
K_o	:	0.50

SUBGRADE:
YOUNGS MODULUS	:	5,000
POISSONS RATIO	:	0.40
UNIT WEIGHT	:	0.06
K_o	:	0.50

Figure 16. Case 1, single-axle load

RAIL

BALLAST   E=25,000 PSI $\nu=0.35$

SUBGRADE   E=5000 PSI $\nu=0.40$

33,000 LB   33,000 LB

70"   12"

BALLAST:
YOUNGS MODULUS	:	25,000 PSI
POISSON'S RATIO	:	0.35
UNIT WEIGHT	:	0.064 PSI
K_o	:	0.50

SUBGRADE:
YOUNGS MODULUS	:	5,000
POISSON'S RATIO	:	0.40
UNIT WEIGHT	:	0.06
K_o	:	0.50

MATERIAL PROPERTIES:

RAIL:
SECTION	:	90 RE-A
AREA	:	8.82 IN.^{2}
SECTION MODULUS (BASE)	:	15.23 IN.^{3}
MOMENT OF INERTIA	:	38.70 IN.^{4}
YOUNGS MODULUS	:	30×10^6 PSI

TIE:
WIDTH	:	9.0 IN.
DEPTH	:	7.0 IN.
LENGTH	:	102.0 IN.
MOMENT OF INERTIA	:	257.25 IN.^{4}
YOUNGS MODULUS	:	1.5×10^6 PSI
CENTER-TO-CENTER SPACING	:	20.0 IN.

Figure 17. Case 2, two-axle truck

RAIL

33,000 LB   33,000 LB   33,000 LB   33,000 LB

70"   80"   70"

33,000 LB   33,000 LB

70"

12"

BALLAST   E=25,000 PSI   ν=0.35

SUBGRADE   E=5000 PSI   ν=0.40

MATERIAL PROPERTIES:

RAIL:
SECTION : 90 RE-A
AREA : 8.82 IN.^{2}
SECTION MODULUS (BASE) : 15.23 IN.^{3}
MOMENT OF INERTIA : 38.70 IN.^{4}
YOUNGS MODULUS : 30x10^6 PSI

TIE:
WIDTH : 9.0 IN.
DEPTH : 7.0 IN.
LENGTH : 102.0 IN.
MOMENT OF INERTIA : 257.25 IN.^{4}
YOUNGS MODULUS : 1.5 x10^6 PSI
CENTER-TO-CENTER SPACING: 20.0 IN.

BALLAST:
YOUNGS MODULUS : 25,000 PSI
POISSON'S RATIO : 0.35
UNIT WEIGHT : 0.064 PSI
K_o : 0.50

SUBGRADE: YOUNGS MODULUS : 5,000
POISSON'S RATIO : 0.40
UNIT WEIGHT : 0.06
K_o : 0.50

Figure 18.   Case 3, two adjacent two-axle trucks

tie length. The number and location of the wheel loads being considered affect the amount of input to each of the models. GEOTRACK can evaluate up to four wheel loads; however, the loads must be located at a tie. In the ILLITRACK model the loads are considered to be symmetrical about the model center line. This requires that only half of the actual wheel or truck configuration be input. For example a 30,000-lb single-wheel load is modeled as a 15,000-lb load at the center line and a two-axle truck (30,000-lb wheel loads with 70-in. center-to-center spacing) is modeled as one 30,000-lb wheel load 35 in. from the center line. The option is also available to input specified deflections instead of loads. KENTRACK will accept up to 25 loads at any location in the model. Various inputs are required for controlling the calculations and printed output of GEOTRACK and KENTRACK as summarized in Table 7. No input data for controlling the printed output are required in ILLITRACK as all of the input and computed data are output.

147. Physical data used as input into the three computer models are almost identical; therefore, the information obtained from field inspection (rail properties, tie properties, and layer thicknesses), field testing (estimated Young's modulus, unit weight, and layer thicknesses), or laboratory testing (resilient modulus, etc.) would be applicable to each of the models. Two parameters that cannot be measured are the effective bearing length, L , and angle of distribution, ϕ , required in the ILLITRACK model. Recommended values are $L = 18$ and $\phi = 10$; however deviation from these empirical values will produce a wide range of results as shown in a later section.

148. _Data preparation and input._ All of the candidate models were run on the University of Kentucky IBM 3081 computer system using batch processing with punched cards as input. Data preparation for each model included making a sketch of the loading conditions, coding the input data on a computer coding form, and keypunching the data onto cards. The time required to prepare the data for keypunching the cards was approximately the same for each of the models. GEOTRACK and KENTRACK required approximately the same amount of time for card punching with GEOTRACK taking slightly longer. The time required to punch a longitudinal case of ILLITRACK was comparable to the time required to punch a test case using GEOTRACK; however, preparing the cards for a complete analysis using both a longitudinal and transverse analysis took approximately 50 percent longer for ILLITRACK than did card punching for GEOTRACK. In order to provide a comparison of the computer models to the BOEF model track modulus

values computed by GEOTRACK and KENTRACK were averaged, and this average track modulus (3,430 psi) was used as input into the BOEF model.

149. **Program run time.** The computer time required to run each of the test cases using each of the candidate models is presented, along with the cost, in Table 8. As seen in this table, GEOTRACK requires approximately 2.5 times more computer time than ILLITRACK and over 8 times more computer time than KENTRACK. ILLITRACK requires approximately 3.3 times more computer time than KENTRACK. It is evident from this comparison that the KENTRACK model runs faster and at a lower cost than either of the other candidate models.

150. **Output.** Table 9 presents a list of the data output from each of the candidate models. An "X" in Table 9 indicates direct output, an "S" indicates that the value must be selected from a list of output data, and a "C" indicates that the value must be calculated from the given data. Comparison of the output from each of the models indicates the KENTRACK model outputs a larger variety of data than either GEOTRACK or ILLITRACK; however, KENTRACK has the limitation of only allowing output of information obtained at the top and bottom of the various layers. This limitation can be overcome by adding extra layers with the same material properties; however, this will increase the amount of input data required. The ILLITRACK program outputs a very large amount of data which includes the displacement and stresses at each node, stresses and strains at each element, and moment at selected elements. This extensive data output is a major drawback to the ILLITRACK program because the user must go through all of the data to select the desired values. As seen in Table 9, the GEOTRACK program is primarily concerned with predicting the response of the ballast and subgrade materials. A limitation to GEOTRACK is the fact that the deflections, stresses, and strains can only be determined at five depths below each tie and that no determinations can be made at the layer boundaries.

151. **Accuracy of o' put.** The true test of the accuracy of any track response model is the comp. ison of that model with actual field data. A full-scale field test program to validate each of the candidate models was beyond the scope of this project; therefore previous validations reported in the literature were used to evaluate the accuracy of the models. Extensive evaluations of GEOTRACK, ILLITRACK, MULTA, and PSA were made by Selig et al. and reported in "A Theory for Track Maintenance Life Prediction" (Aug 1979, 1981). A limited evaluation of ILLITRACK is reported in "Structural Model and

Materials Evaluation Procedures" (Robnett et al. 1976). Selig et al. (1981) used dynamic measurements from five instrumented track sections at Facility for Accelerated Service Testing (FAST) for comparison of these models. Using these field data, these researchers showed that predictions obtained from GEOTRACK were closer to the measured values than predictions obtained from either PSA or ILLITRACK. They concluded that the GEOTRACK model produced results that were in reasonable agreement with measured field results (Selig et al. 1981). From analysis of these reported evaluations it appears that GEO-TRACK comes the closest of the models compared to predicting the actual track response. To evaluate the candidate response models in this study, GEOTRACK was assumed to more nearly represent the actual track response and was used as the basis for comparison. The results of the BOEF analysis were also used in the comparison of the models.

152. <u>Accuracy of output, test case 1.</u> Figures 19 through 24 present the results of test case 1 in terms of rail moment, rail deflection, ballast pressure, subgrade pressure, subgrade deflection, and vertical stress distribution with depth for each of the track response models. Two sets of data are shown for ILLITRACK. One group, denoted run 1, presents the results determined using $L = 18$ in. and $\phi = 10$ deg as recommended in the user's guide. The second group, denoted run 2, presents results determined using $L = 24$ in. and $\phi = 20$ deg as recently recommended in conversations with the program developers. Further discussion of the effect of the L and ϕ terms on the ILLITRACK results is presented in paragraph 155. Figure 19 presents a plot of rail moment versus tie number. Only one data point was calculated by the GEOTRACK model for the single-axle load, and this point falls almost exactly on the data point determined by KENTRACK at tie 1. This plot indicates that GEOTRACK and KENTRACK calculate essentially the same rail moment at the point of load application. The moment calculated by KENTRACK is less than the moment calculated using ILLITRACK or BOEF analysis for the first two ties, is approximately the same at ties 3 and 4, and is again slightly less at tie 5. ILLITRACK run 1 results plot fairly close to the BOEF results for ties 1 and 2, although the negative moment calculated using BOEF is greater at ties 3, 4, and 5 than either of the other models. Comparison of the predicted rail deflections plotted in Figure 20 indicates that the GEOTRACK and KENTRACK results are almost identical. The BOEF results are nearly identical to GEOTRACK and KENTRACK at the load point but drop to a value near the ILLITRACK run 2
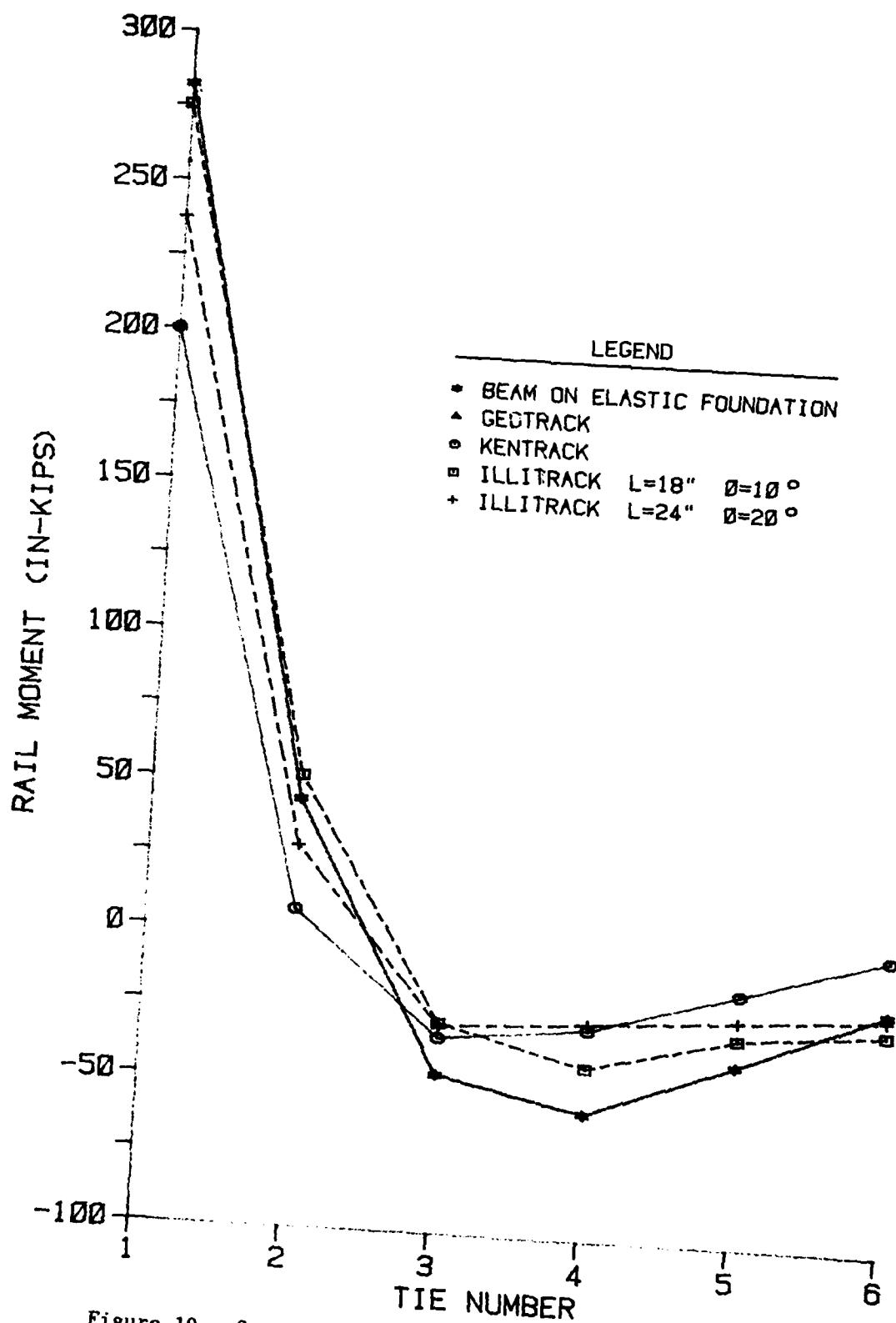
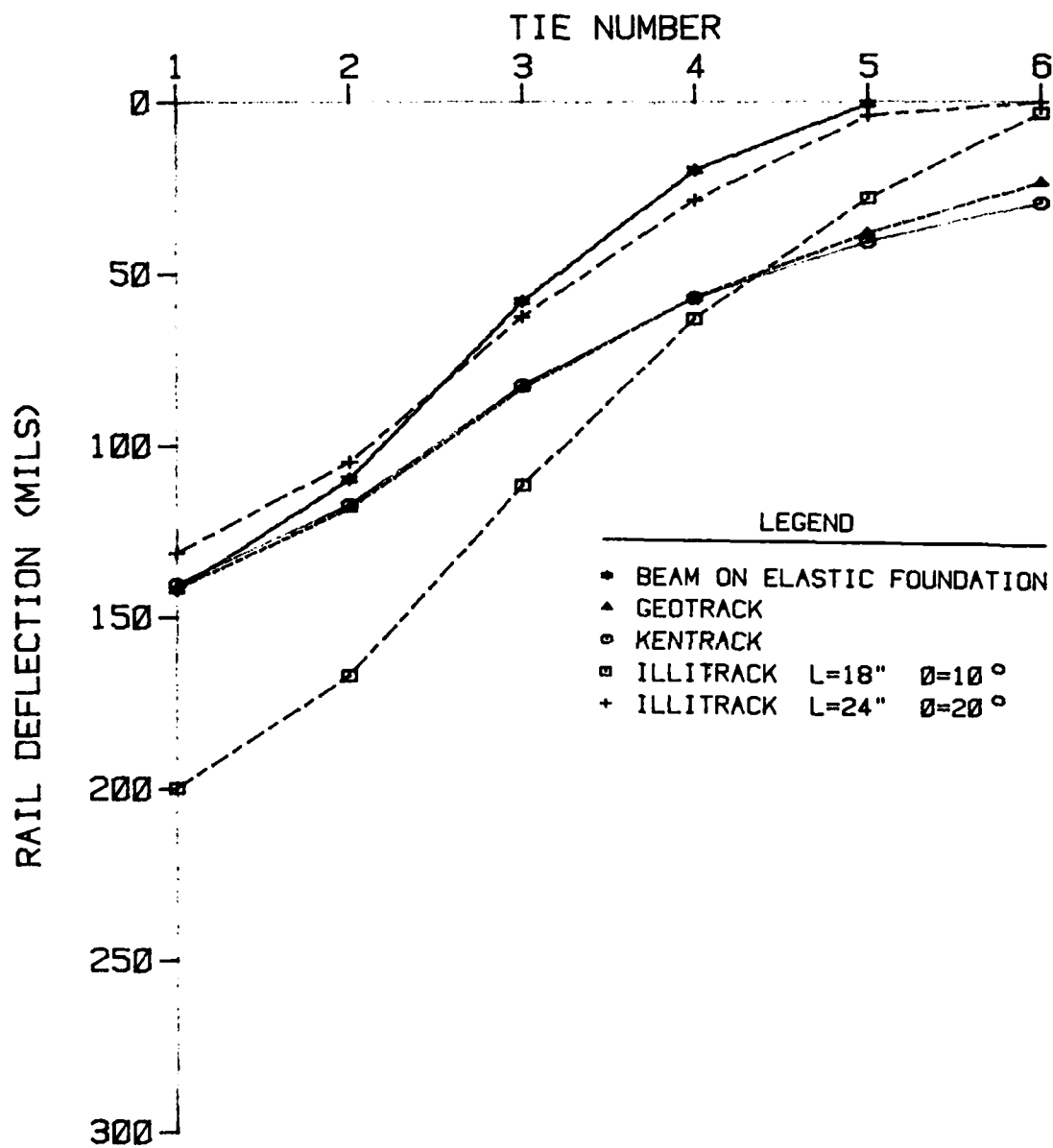Figure 19. Comparison of rail moment predictions, case 1

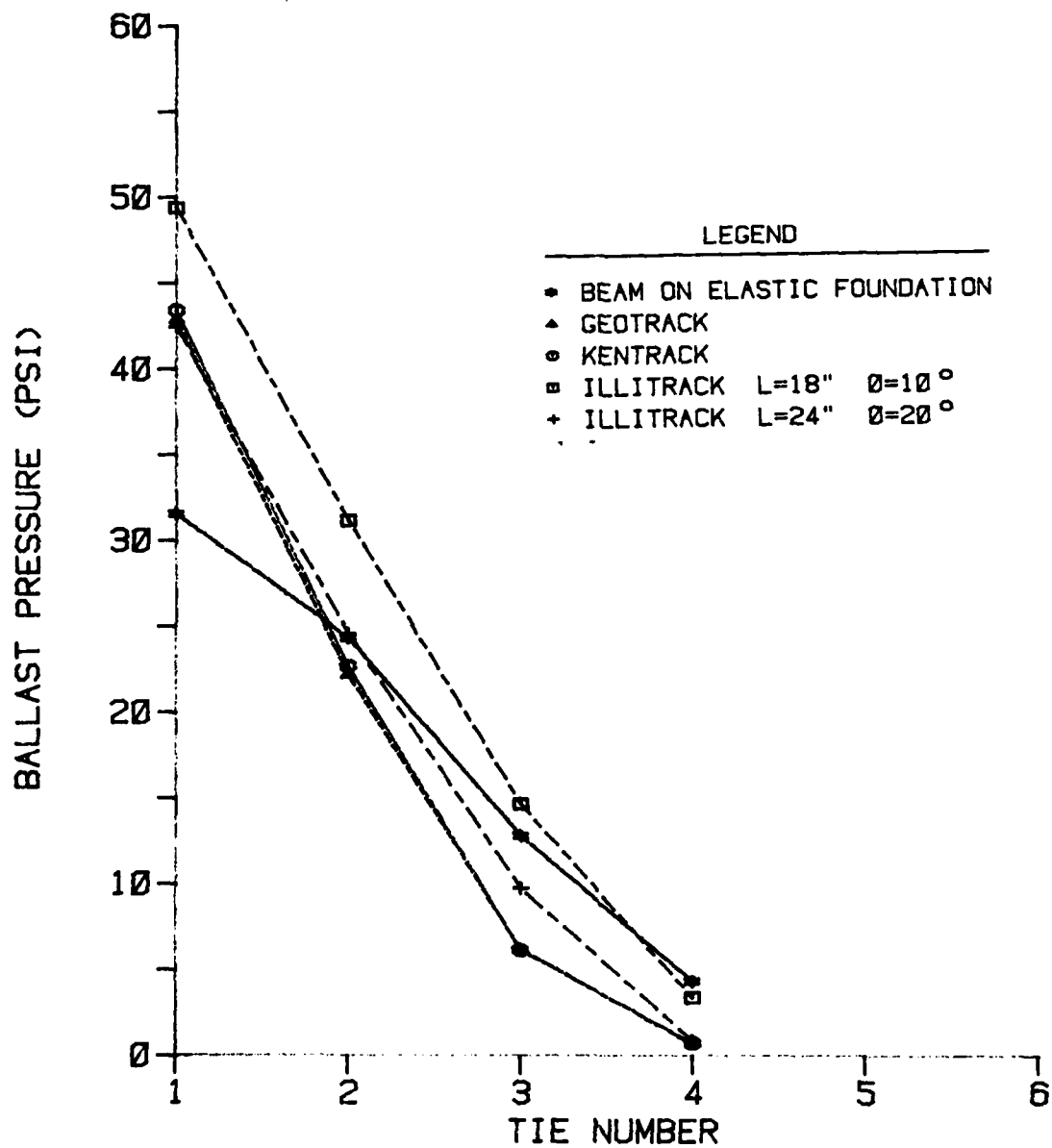Figure 20.   Comparison of rail deflection predictions, case 1

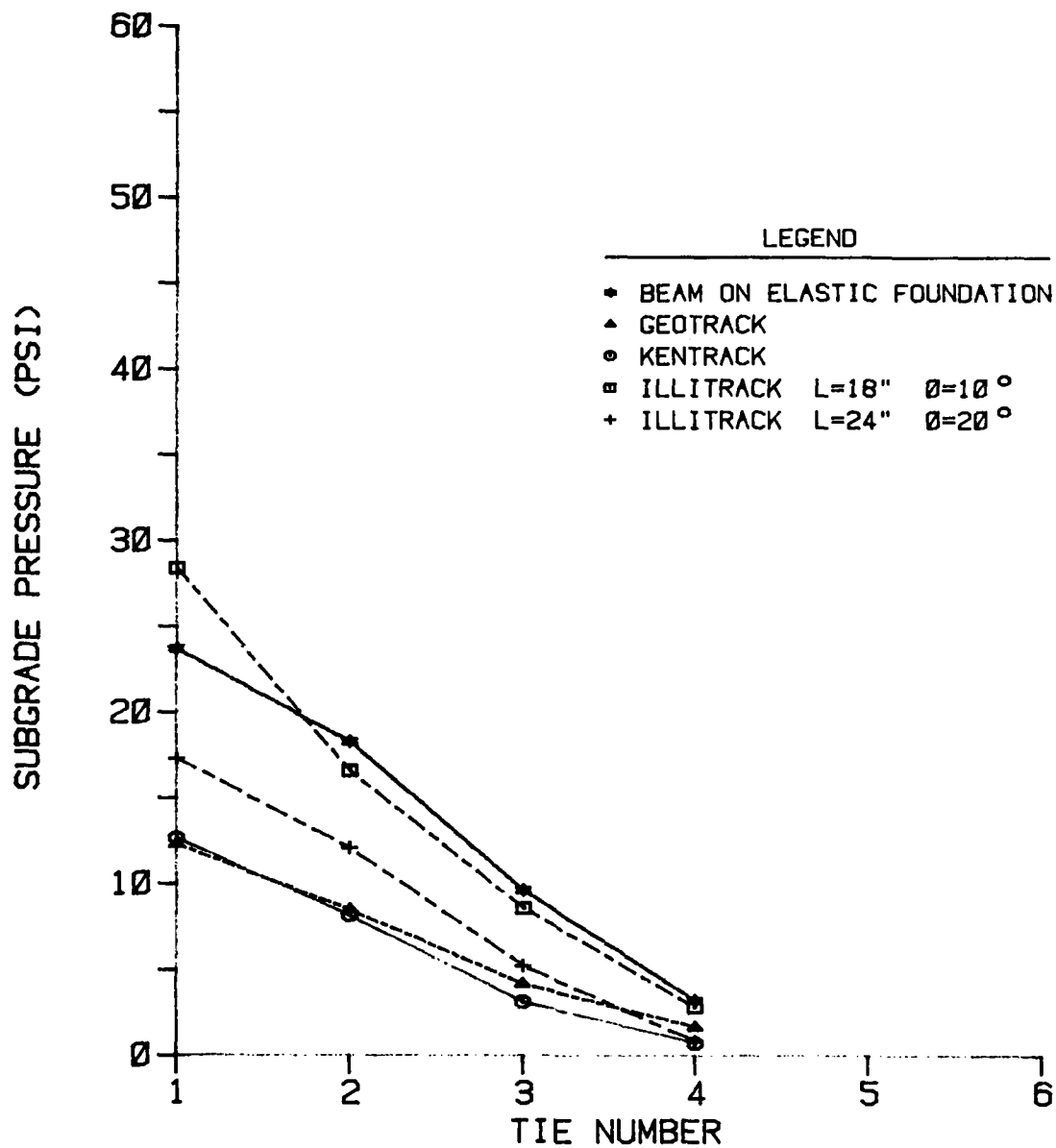Figure 21.   Comparison of ballast pressure predictions, case 1

Figure 22.   Comparison of subgrade pressure predictions, case 1
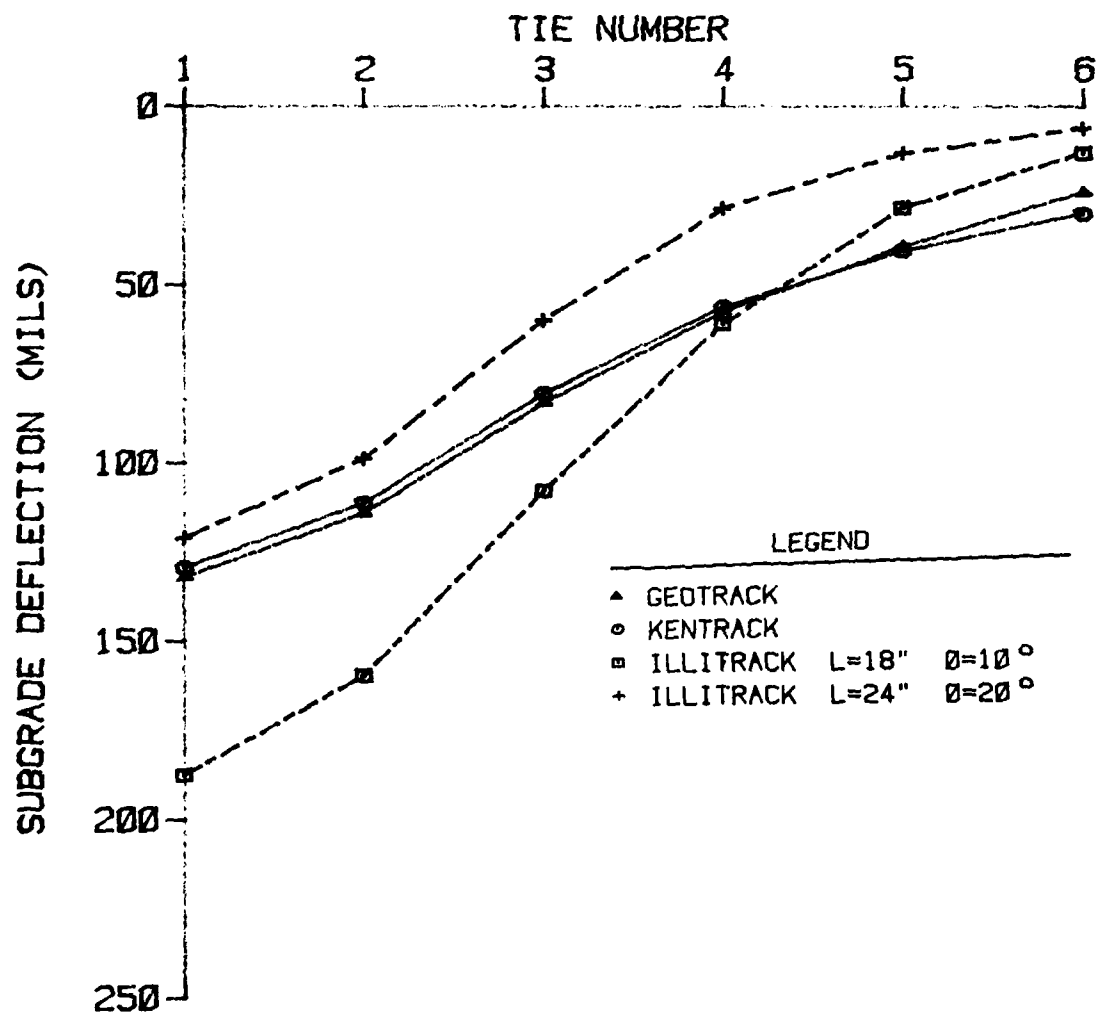
Figure 23.   Comparison of subgrade deflection predictions, case 1
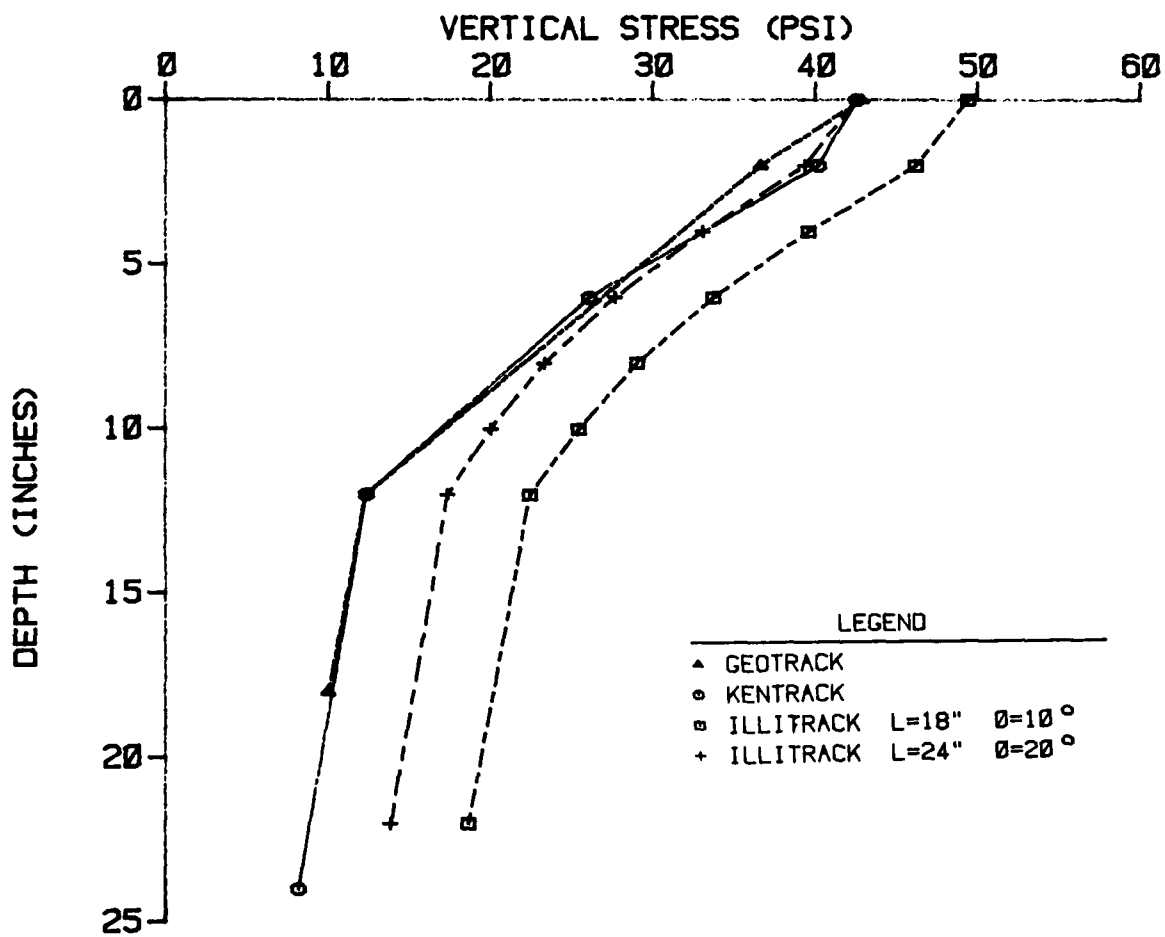
Figure 24.   Comparison of vertical stress distributions, case 1

results from ties 2 through 5. The results of ILLITRACK run 1 are significantly different from the results of either GEOTRACK or the BOEF analysis. Figure 21 compares the ballast pressure predicted by the various models. Once again GEOTRACK and KENTRACK yielded almost identical results, with the ILLITRACK run 2 predictions being fairly close. The ILLITRACK run 1 is again higher than the other models. The BOEF predictions cut across the other predictions yielding a lower prediction at the load point and slightly higher prediction at tie 4. Comparison of the subgrade pressures in Figure 22 once again indicates almost identical predictions using GEOTRACK and KENTRACK. The ILLITRACK run 1 predictions compare best with the BOEF analysis, although the pressure predicted under the loaded tie using BOEF analysis is lower than the pressure predicted with ILLITRACK. Predictions of the subgrade deflection presented in Figure 23 indicate that GEOTRACK and KENTRACK yield almost identical results, while the ILLITRACK runs show a considerable difference in predicted deflection. The vertical pressure distribution with depth under the rail seat of tie 1 was plotted in Figure 24. Once again KENTRACK and GEOTRACK predicted almost identical results. The results of ILLITRACK run 2 from 0 to 8 in. are very similar to the results obtained with GEOTRACK and KENTRACK, but below 8 in. the predicted pressure is larger. The pressures predicted with ILLITRACK run 1 are consistently larger than the other predictions.

153. _Accuracy of output, test case 2._ Figures 25 through 29 present the comparisons of rail moment, rail deflection, ballast pressure, subgrade pressure, and subgrade deflection for test case 2 runs of the candidate track response models. In this and subsequent ILLITRACK runs only L = 18 in. and ϕ = 10 deg were as recommended in the user's guide. Figure 25 presents a plot of predicted rail moment versus tie number. At tie 1 the rail moment predicted by GEOTRACK is somewhat lower than the rail moment predicted by either KENTRACK or BOEF. At tie 2 the BOEF and GEOTRACK predictions were very close with the KENTRACK prediction somewhat higher. At tie 3 the BOEF prediction was more negative than the GEOTRACK and ILLITRACK predictions. The KENTRACK prediction was slightly less negative than the GEOTRACK prediction. The BOEF, GEOTRACK, and KENTRACK predictions were all very close for ties 4 and 5. The GEOTRACK and KENTRACK predictions for ties 6 through 10 were in very good agreement while the BOEF predictions were larger. Additional use of the KENTRACK model resulted in the discovery of an error in the rail moment calculations for multiaxle cases. This error was due to a programming

91
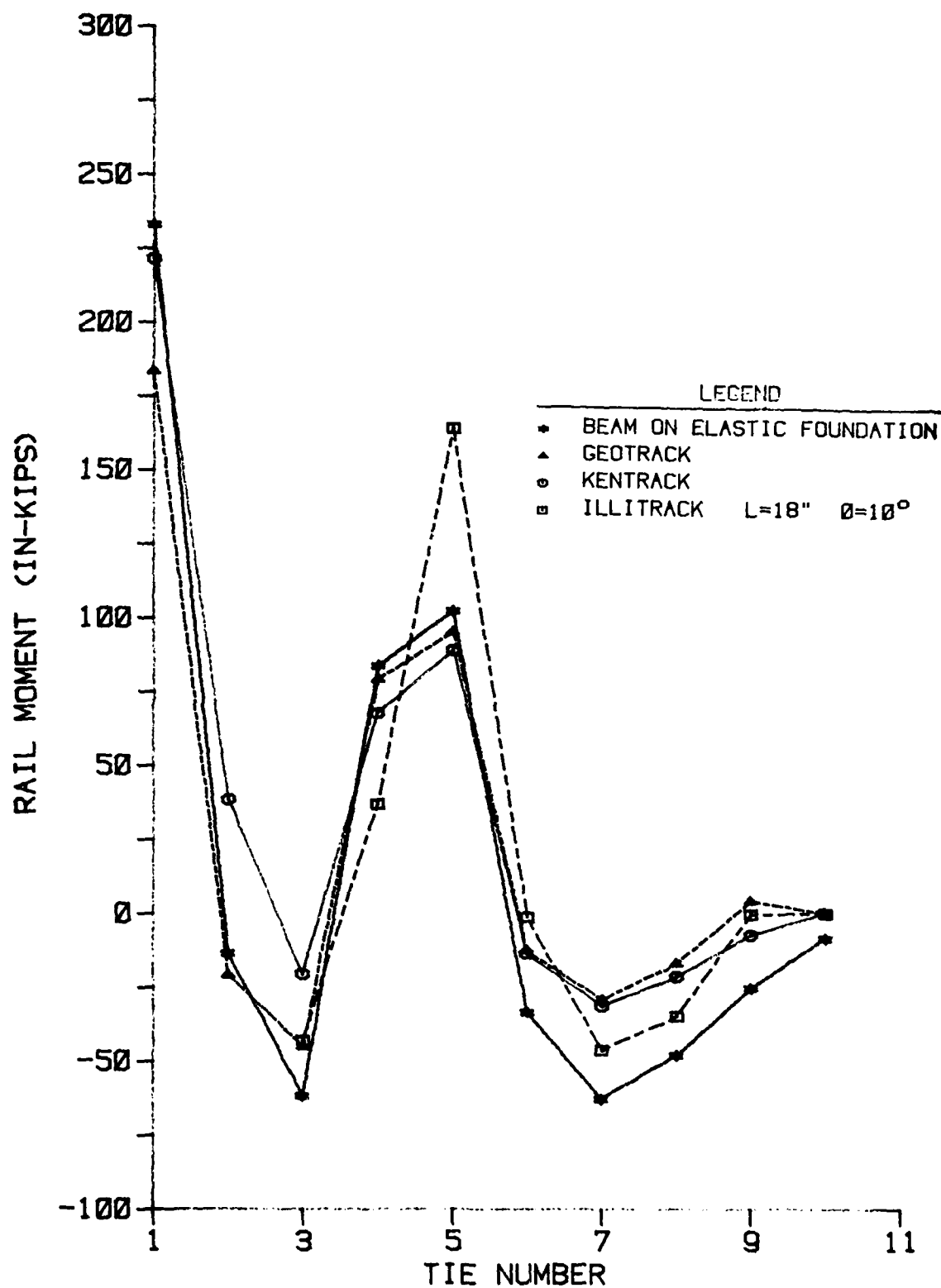
Figure 25.   Comparison of rail moment, case 2

END

1.0

1.1

1.25 1.4 1.6

4.5
5.0
3.6
6.3

2.8
3.2
3.6
4.0

2.5
2.2
2.0
1.8

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

TIE NUMBER

RAIL DEFLECTION (MILS)

LEGEND

- BEAM ON ELASTIC FOUNDATION
- ▲ GEOTRACK
- ● KENTRACK
- ▫ ILLITRACK   L=18"   ∅=10°
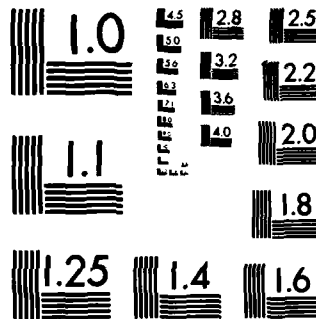
Figure 26.   Comparison of rail deflection, case 2

Figure 27. Comparison of ballast pressure, case 2
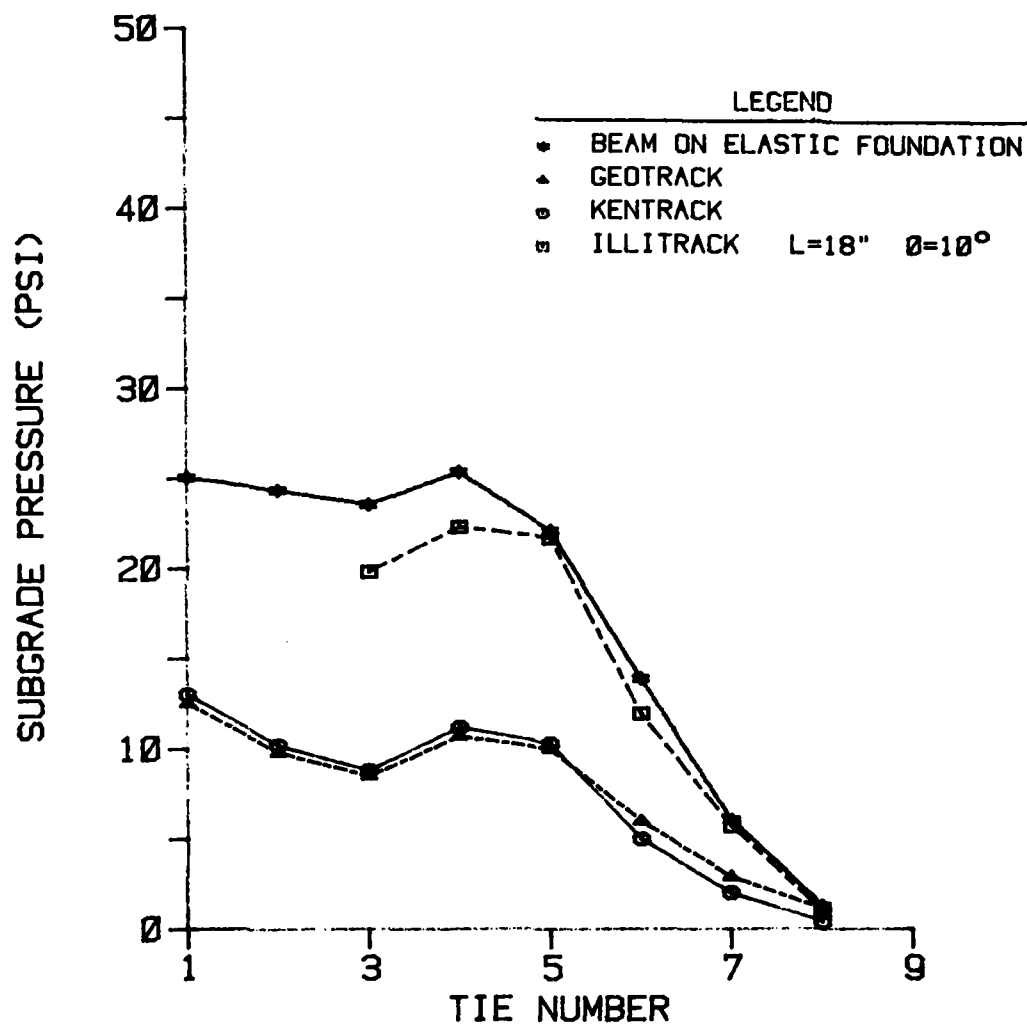
Figure 28.   Comparison of subgrade pressure, case 2
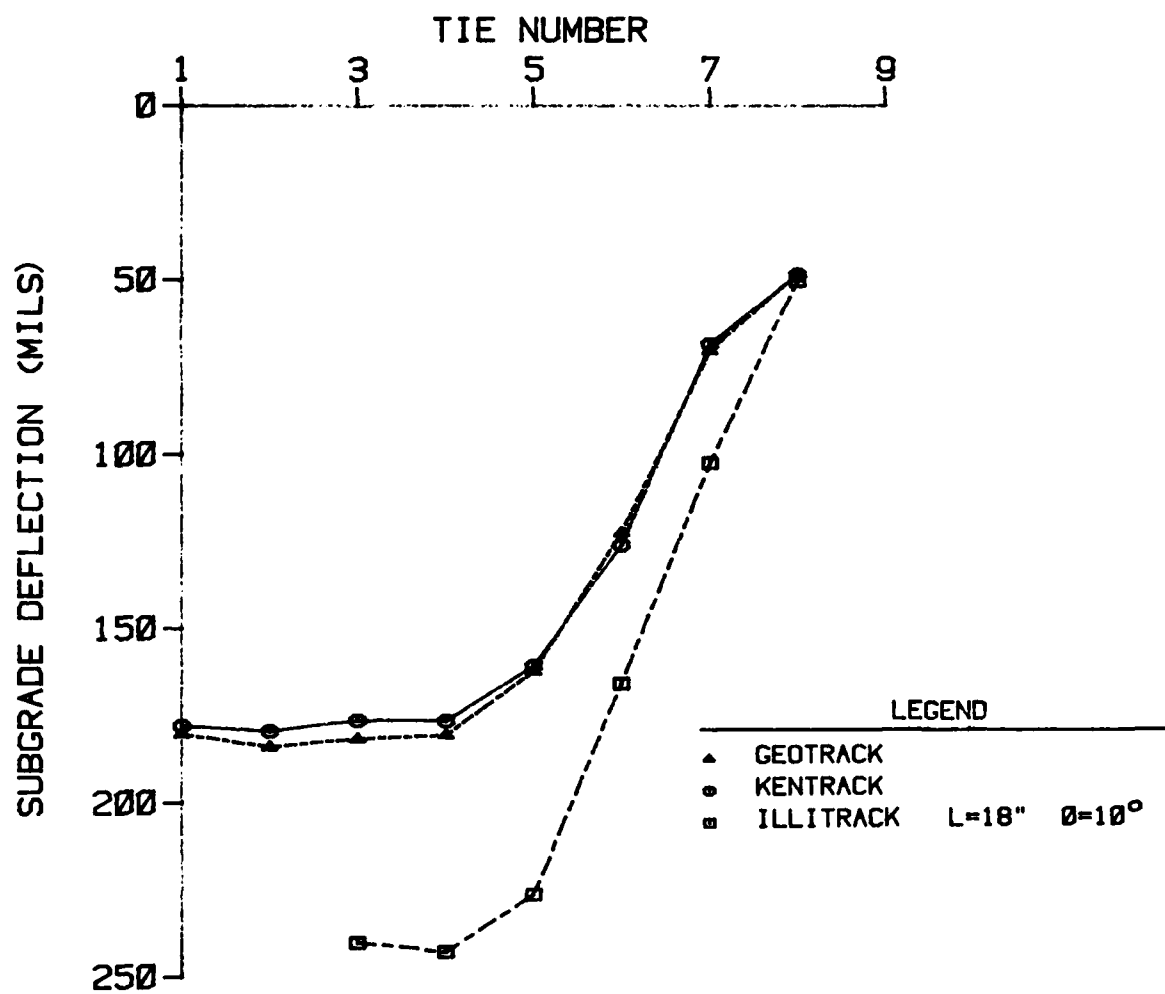
Figure 29.   Comparison of subgrade deflection, case 2

problem and is presently being corrected by the developers. Because of the
loading geometry required by ILLITRACK the first output was available at
tie 3. The resulting rail moments predicted by ILLITRACK were fairly close to
those predicted by the other track response models. Comparisons of the pre-
dicted rail deflections in Figure 26 indicate that KENTRACK and GEOTRACK once
again yield almost identical results. The ILLITRACK deflections were gen-
erally higher than any of the other model predictions. BOEF-predicted deflec-
tions were somewhat smaller than the deflections predicted with GEOTRACK.
Comparison of predicted ballast pressures in Figure 27 indicates that
KENTRACK-predicted pressures were almost identical to GEOTRACK and fairly
close to the BOEF predictions. Again, the ILLITRACK predictions were larger
than either of the other models. As seen in Figure 28, the subgrade pressures
predicted with KENTRACK were very close to the GEOTRACK predictions, and the
ILLITRACK predictions were fairly close to the BOEF predictions. Figure 29
presents a comparison of the predicted subgrade deflections. Once again KEN-
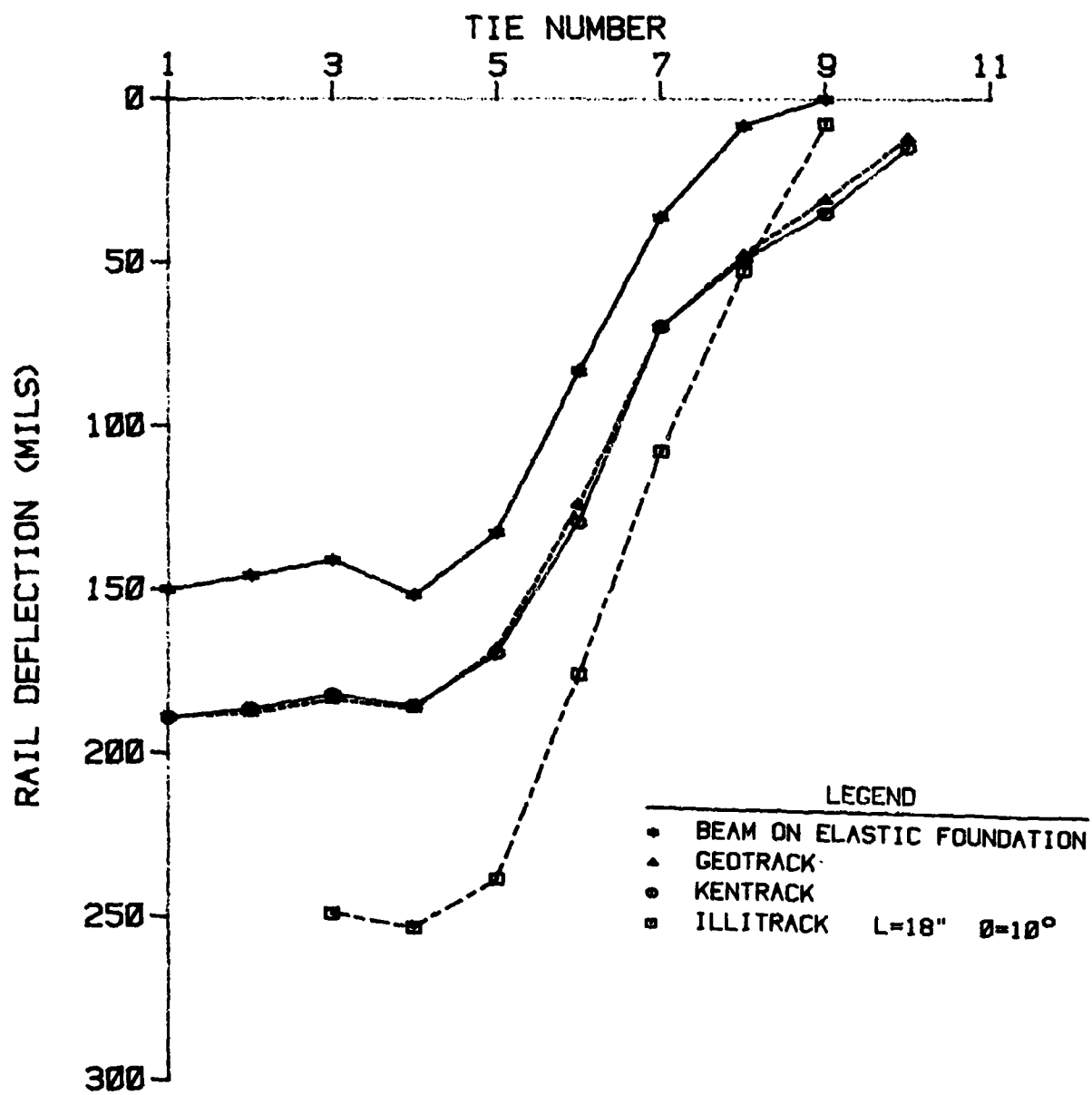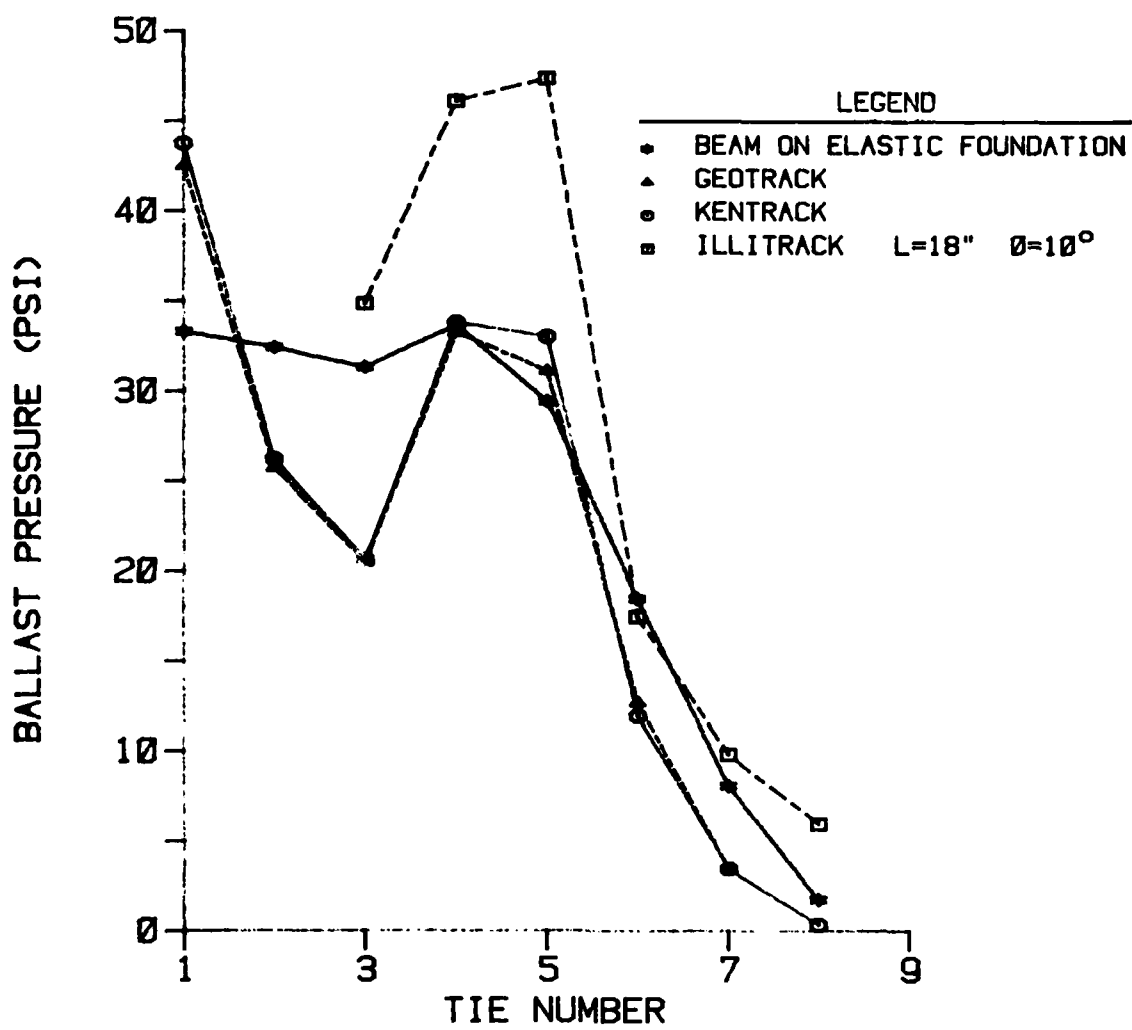TRACK and GEOTRACK yielded almost identical results with the ILLITRACK predic-
tions being larger.

154. <u>Accuracy of output, test case 3.</u> Figure 30 shows the loading con-
ditions used to simulate the test case loadings. Because of the assumption of
a symmetrical loading condition in the ILLITRACK model, only the data from
ties 7 through 15 are output. To facilitate analysis of the data, symmetry
was assumed and the results calculated from ties 7 through 12 were applied to
ties 6 through 1, respectively. Figures 31 through 35 present comparisons of
rail moment, rail deflection, ballast pressure, subgrade pressure, and sub-
grade deflection for test case 3. Because of the restriction on the number
of loads that may be used in a GEOTRACK analysis, two runs of GEOTRACK were
made. Analysis of Figures 31 through 35 and comparison with the results of
the BOEF analysis indicate that GEOTRACK run 1 more nearly simulates the ex-
pected track response at ties 1 through 6, while GEOTRACK run 2 better simu-
lates the response in ties 7 through 15. This limitation on the loading
geometry is a major drawback to the GEOTRACK model. Study of the rail moments
plotted in Figure 31 indicates that the moments predicted by all of the models
are similar with the maximums and minimums varying depending upon the loading
geometry. As discussed in paragraph 153 the rail moments predicted by KEN-
TRACK for multiple loads were erroneous therefore the KENTRACK moments were
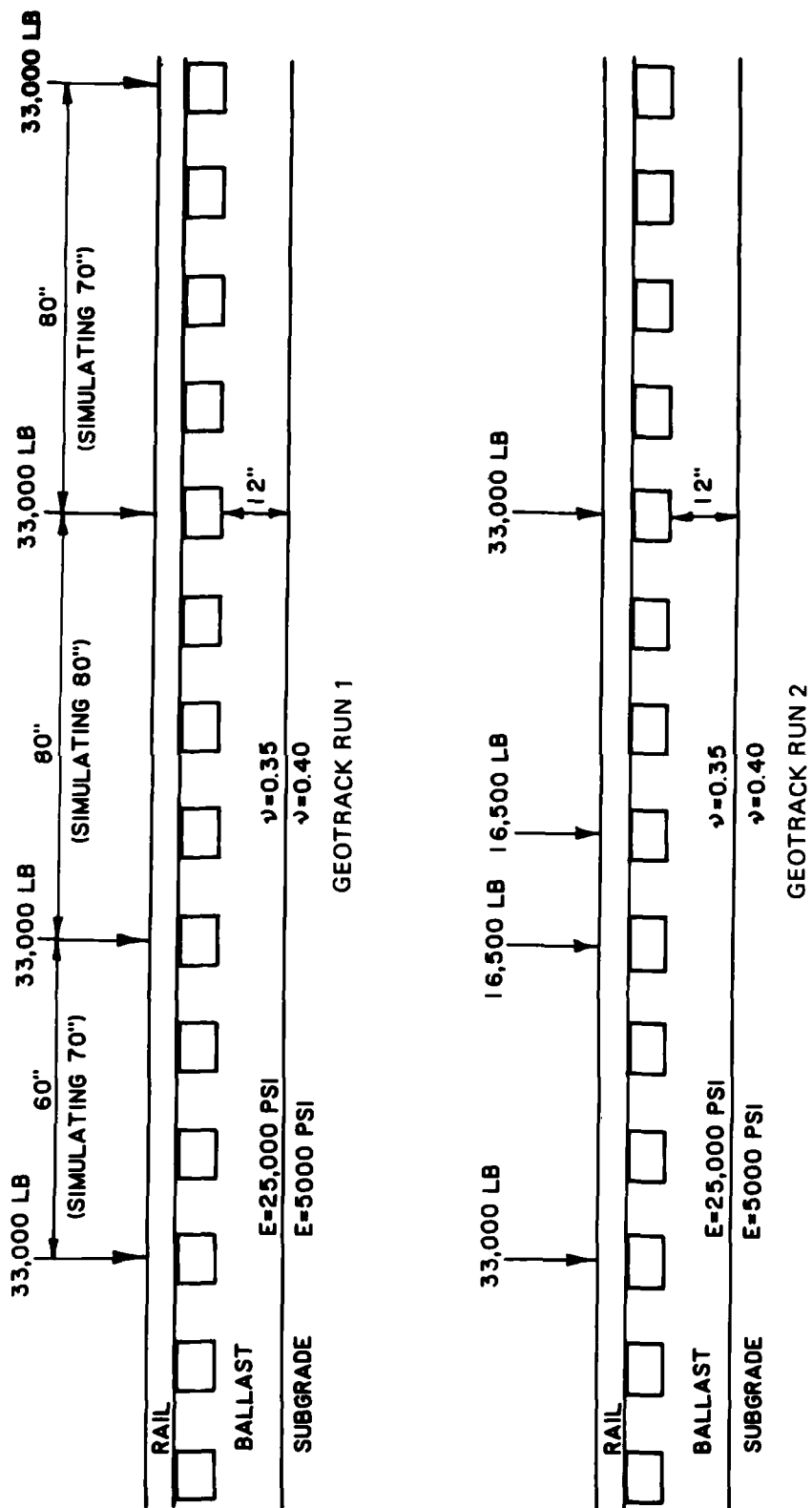not plotted in Figure 31. Rail deflections are compared in Figure 32. It is

97

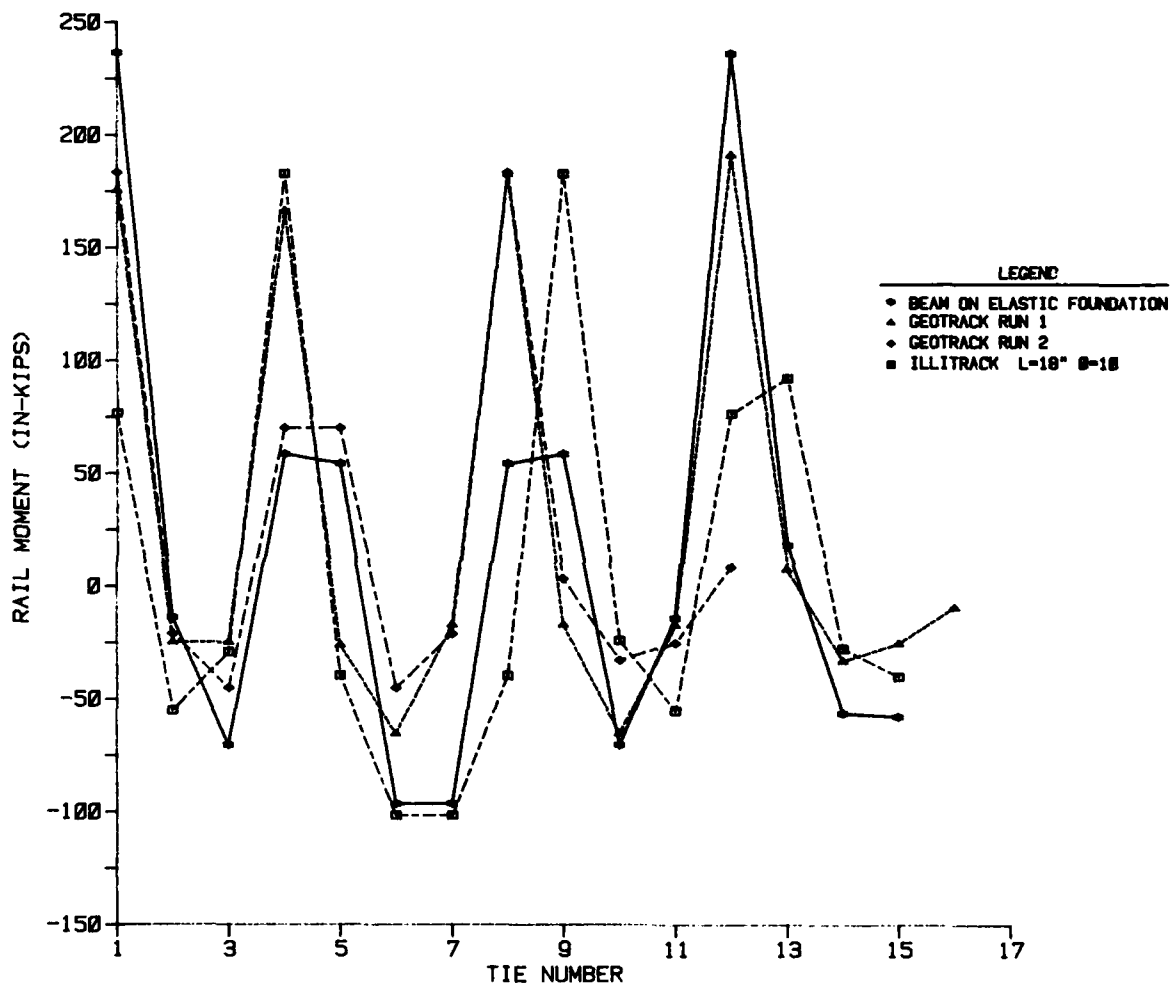Figure 30. Loading conditions for geotrack, case 3

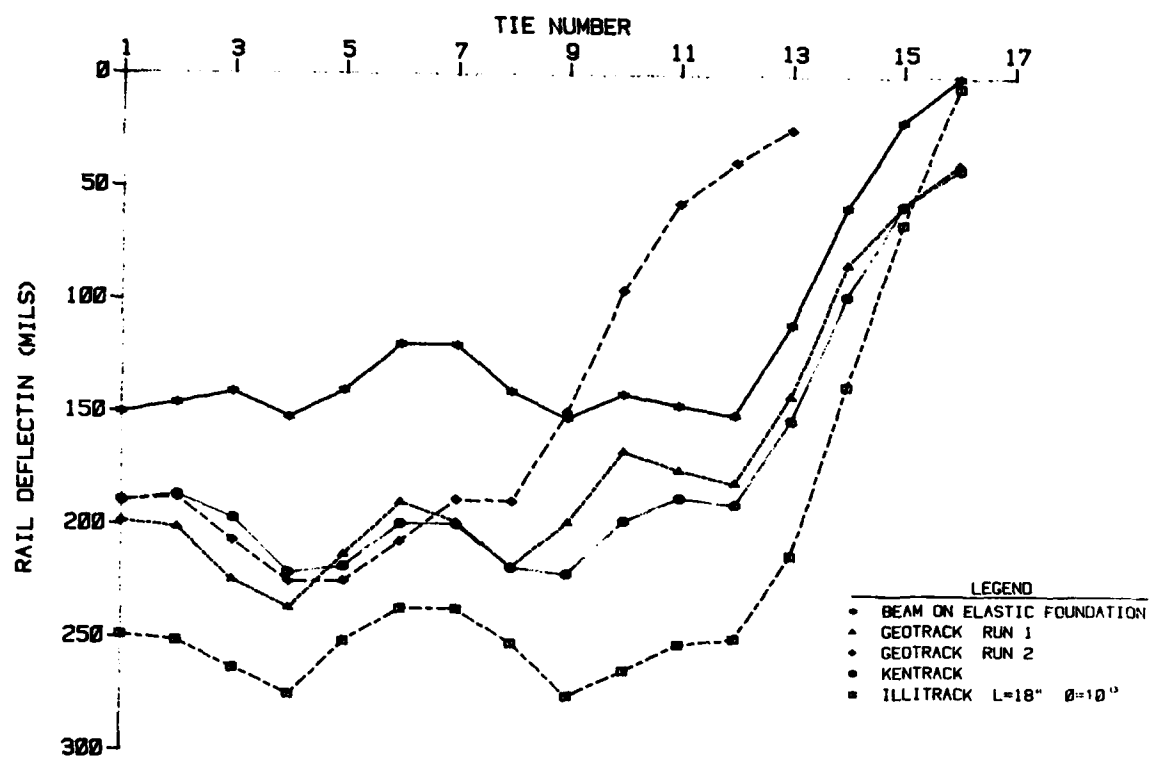Figure 31. Comparison of rail moment, case 3
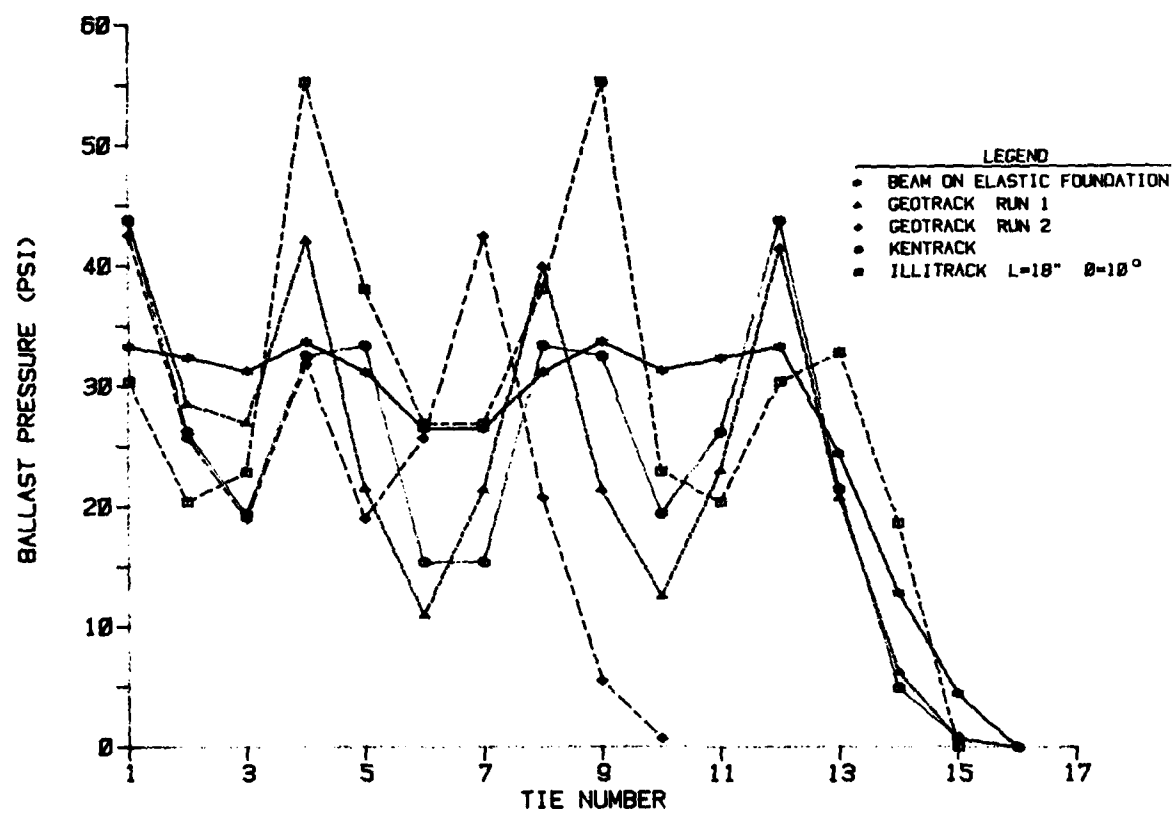
Figure 32. Comparison of rail deflection, case 3

100

Figure 33.   Comparison of ballast pressure, case 3

101
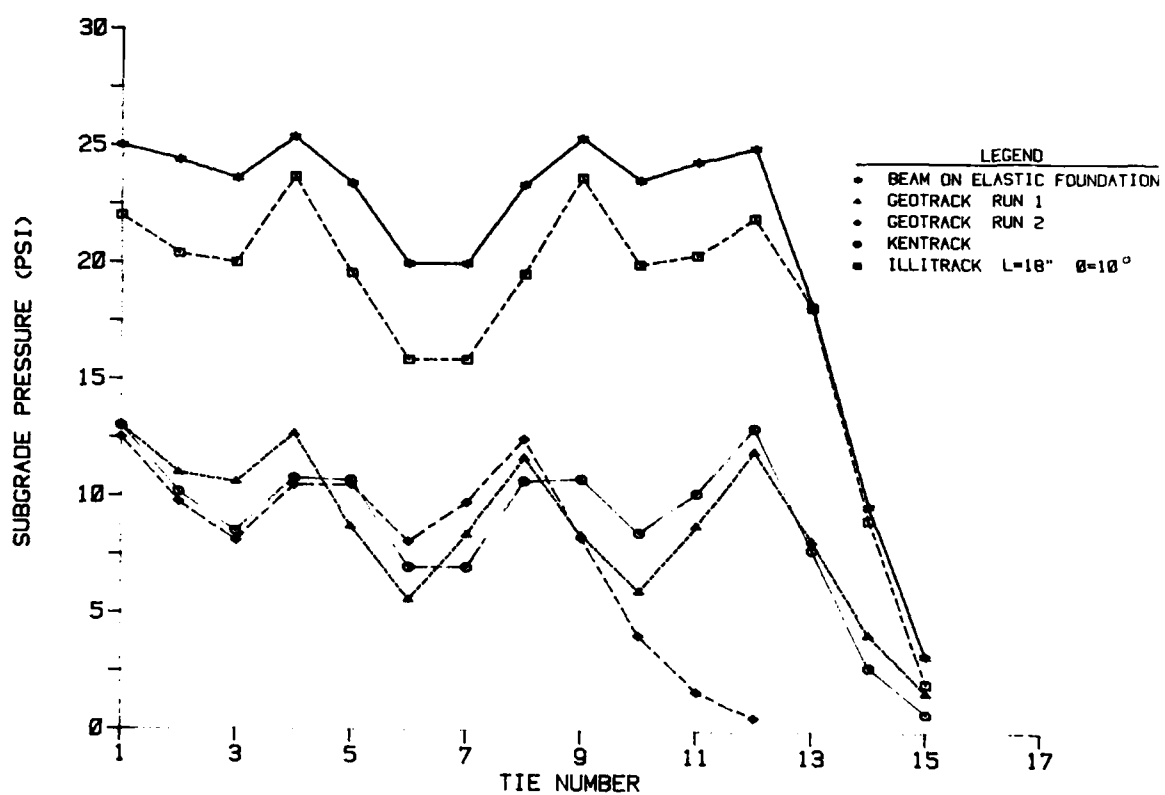
Figure 34.   Comparison of subgrade pressure, case 3
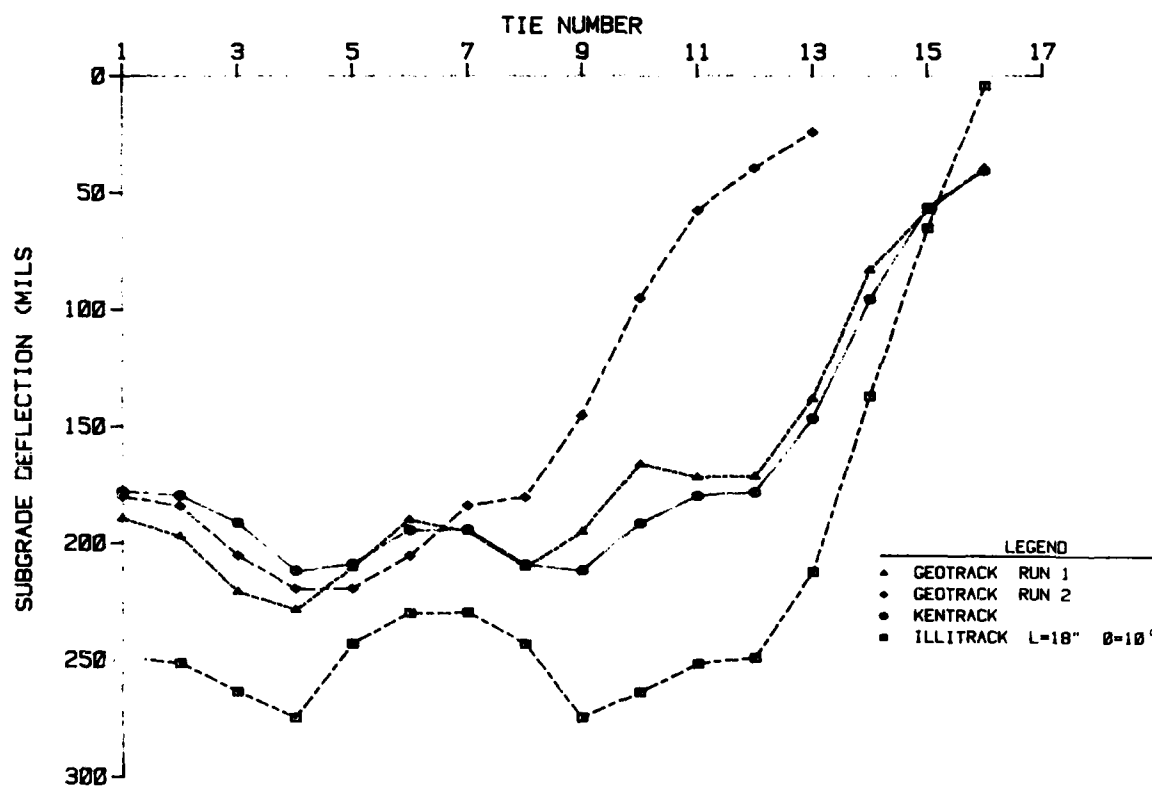
Figure 35.   Comparison of subgrade deflection, case 3

evident that the KENTRACK results agree well with the GEOTRACK results. Although the ILLITRACK-predicted deflections are somewhat larger than deflections predicted with the other models, the shape of the deflection basin is very similar to that of the BOEF analysis and KENTRACK models. Comparisons of the ballast pressure in Figure 33 indicate that the KENTRACK predictions agree fairly well with the GEOTRACK predictions. None of the computer models agrees very well with the BOEF analysis. The ILLITRACK results do not agree well with either GEOTRACK or BOEF analysis. Plots of predicted subgrade pressures (Figure 34) indicate good comparison between the results of GEOTRACK and KENTRACK. The results from ILLITRACK are slightly less than the BOEF results, although the shapes of the curves are similar, indicating a good comparison. Comparison of the predicted subgrade deflections (Figure 35) once again show good comparison between GEOTRACK and KENTRACK results, while the ILLITRACK-predicted deflections were slightly higher.

155. <u>Accuracy of output, effect of L and ϕ on ILLITRACK results.</u>
Several runs of ILLITRACK were made in order to study the effect of varying the effective tie bearing length, L , and angle of distribution, ϕ . Values of L and ϕ used in this comparison are presented in Table 10. Comparison of the rail moments in Figure 36 indicates that ILLITRACK run E predicted moments nearest the GEOTRACK prediction while run A gave results nearest the BOEF prediction at ties 1 and 2. The predicted rail deflections and subgrade deflections plotted in Figures 37 and 38, respectively, show a great deal of variability depending on the parameters of L and ϕ used. In both of these comparisons the results of ILLITRACK run B correspond the best to the GEOTRACK predictions for ties 1 through 3 with runs C and D comparing best at ties 4 through 6. The predicted ballast pressures and subgrade pressures are plotted in Figures 39 and 40, respectively, for the various ILLITRACK runs. As with the deflections, a large amount of variation is evident depending on the parameters used. In comparing the predicted ballast pressures, the ILLITRACK run E is closest to the GEOTRACK results at tie 1, while the results of runs D and F are closer to the GEOTRACK results at ties 2, 3, and 4, respectively. The ballast pressure predicted using the BOEF analysis does not compare well with any of the other models; therefore no detailed comparison was attempted. The subgrade pressures predicted using BOEF analysis compared very well to the results of ILLITRACK run C at tie 1 and ILLITRACK run A at ties 2 through 4. Results of ILLITRACK run F were slightly larger than the GEOTRACK results.
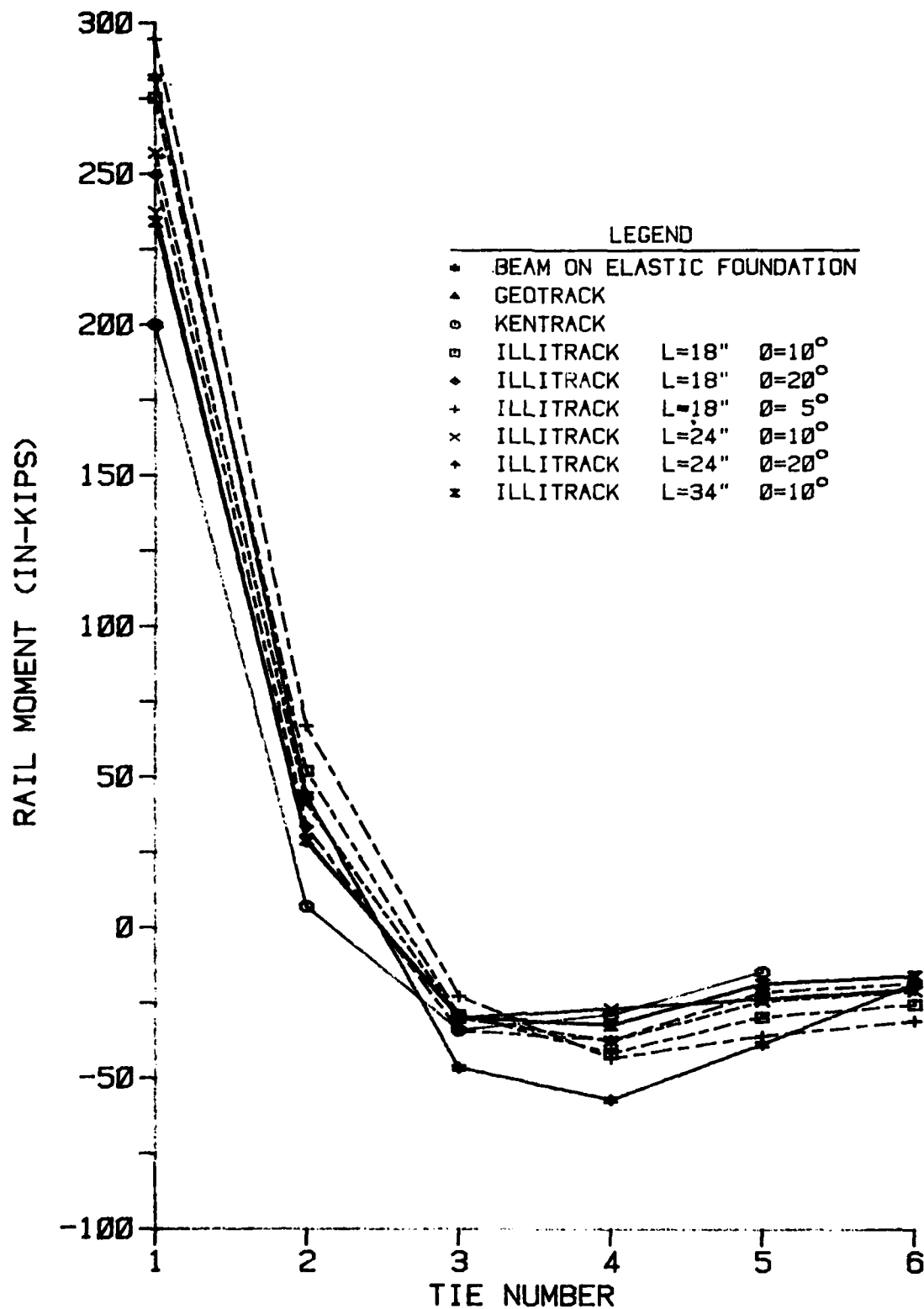
Figure 36. Comparison of ILLITRACK runs, rail moment
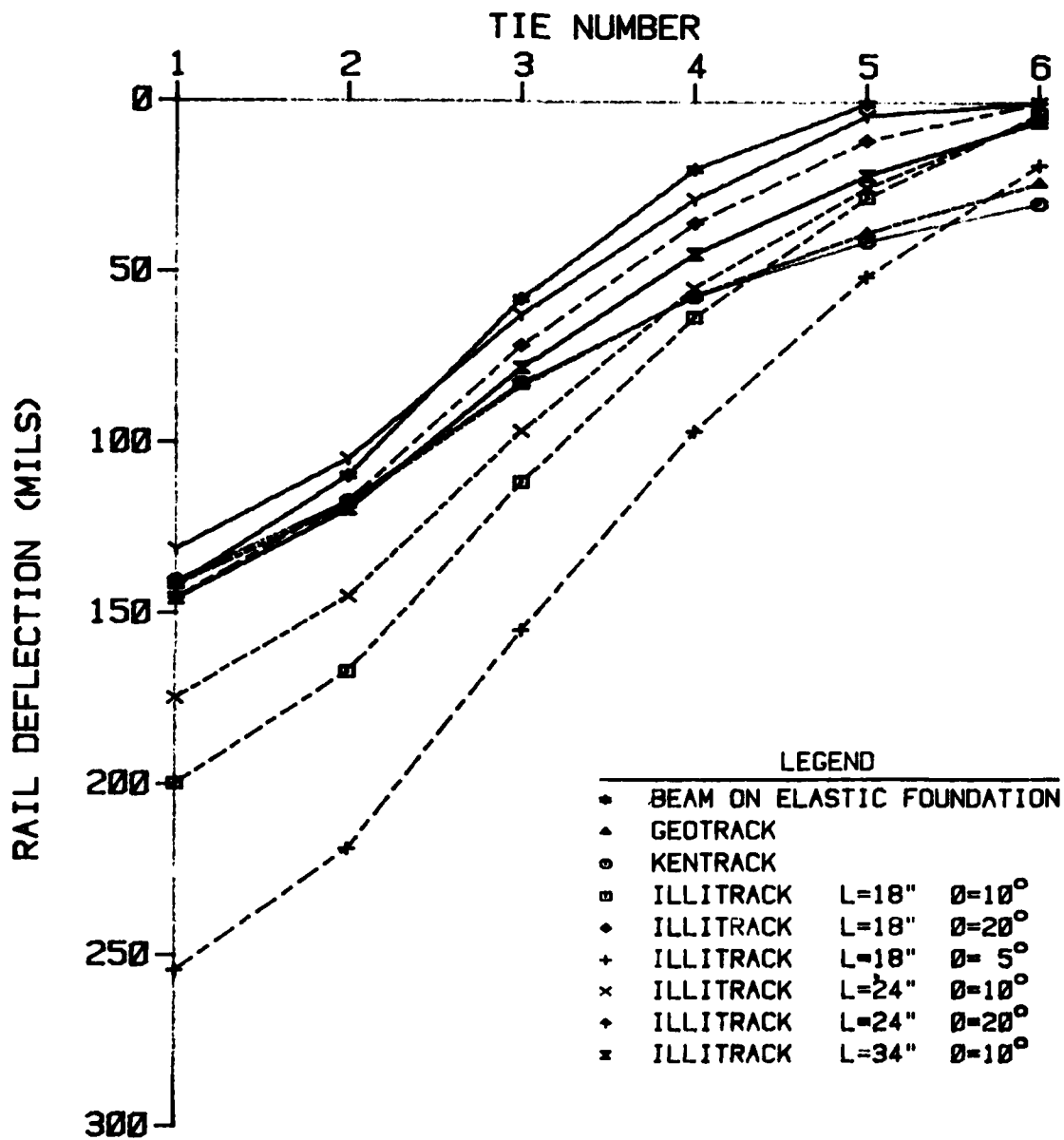
Figure 37. Comparison of ILLITRACK runs, rail deflection

TIE NUMBER

1   2   3   4   5   6

SUBGRADE DEFLECTION (MILS)

0

50

100

150

200

250

LEGEND

▲	GEOTRACK		
☉	KENTRACK		
▣	ILLITRACK	L=18"	Ø=10°
◆	ILLITRACK	L=18"	Ø=20°
+	ILLITRACK	L=18"	Ø= 5°
×	ILLITRACK	L=24"	Ø=10°
✦	ILLITRACK	L=24"	Ø=20°
✖	ILLITRACK	L=34"	Ø=10°
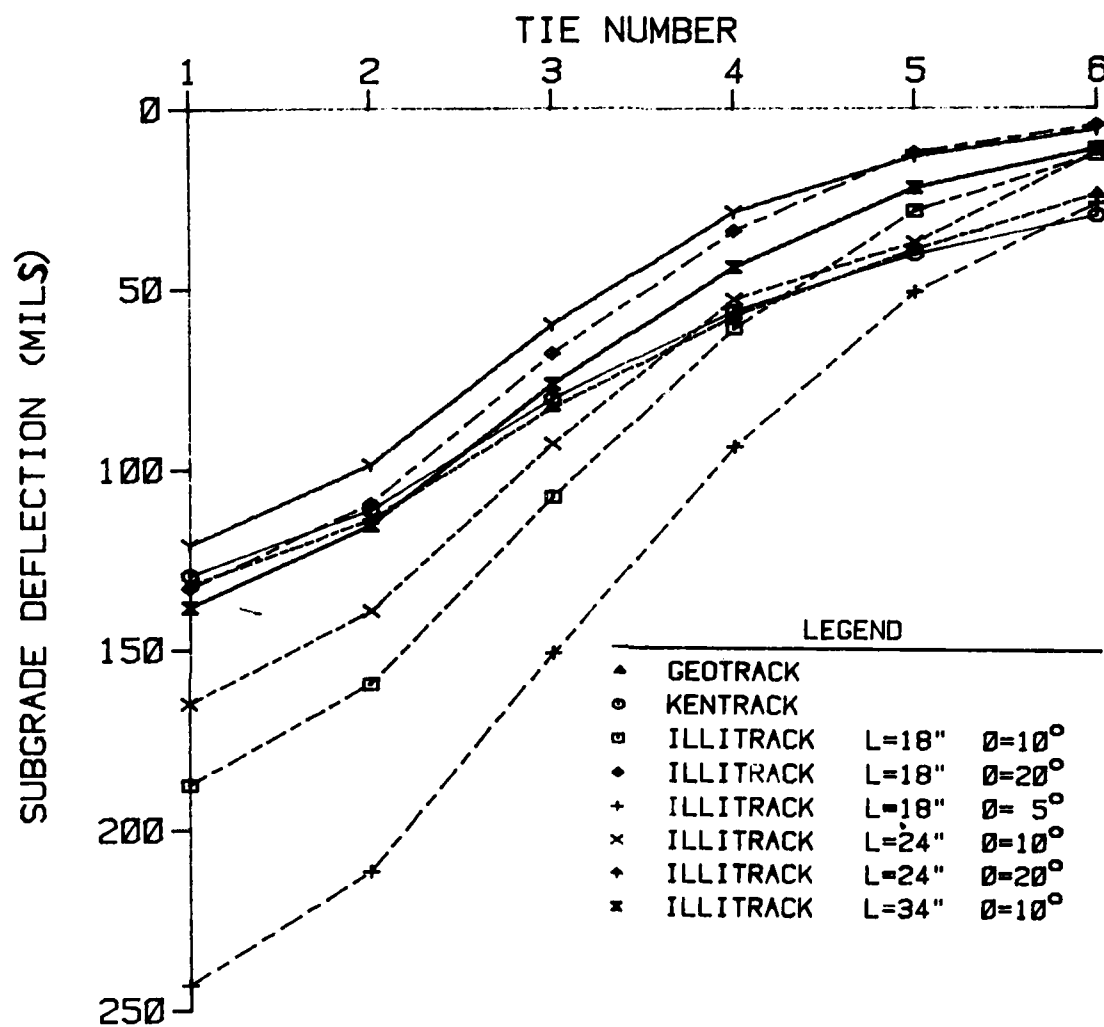
Figure 38.   Comparison of ILLITRACK runs, subgrade deflections

107
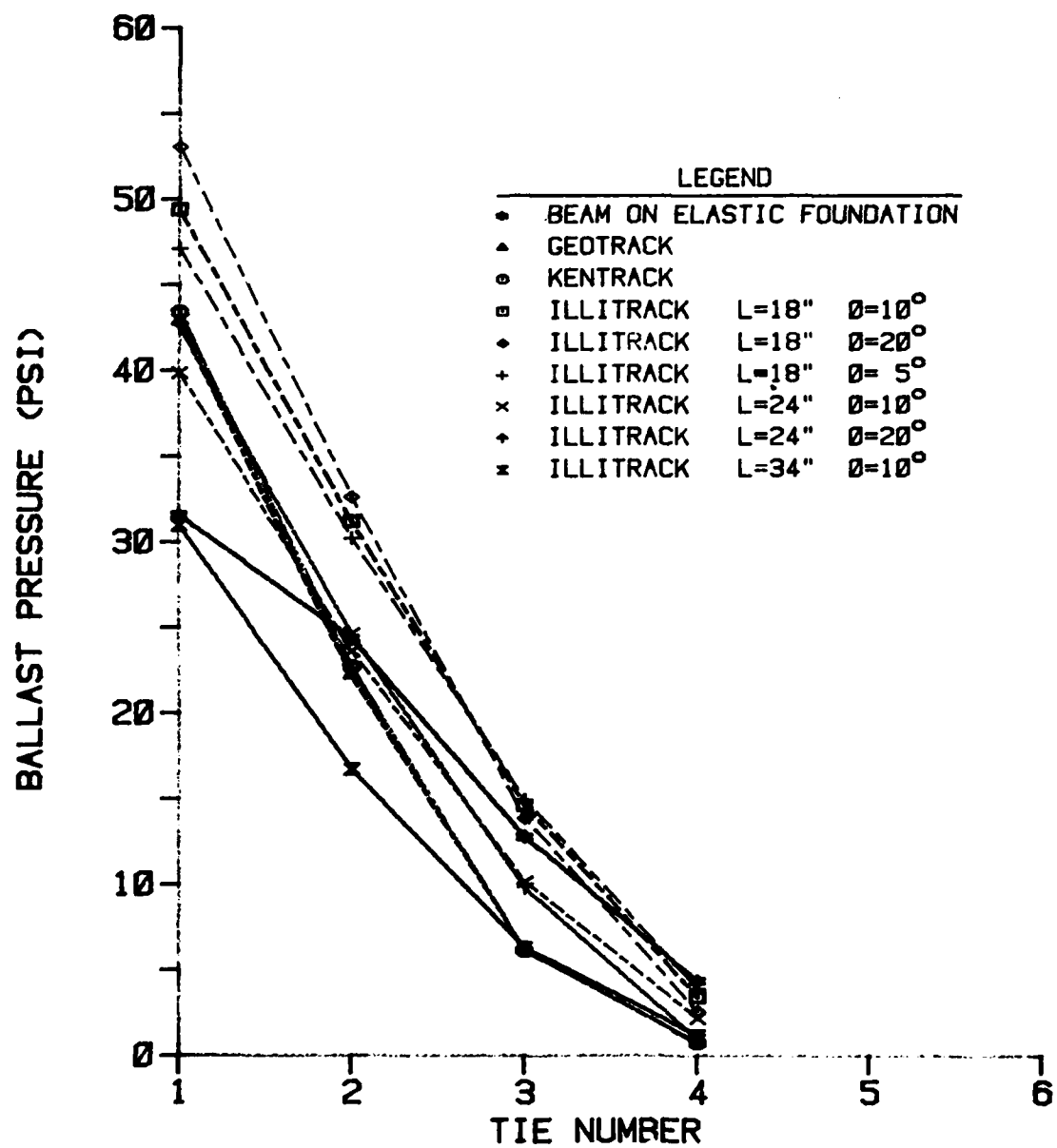
Figure 39. Comparison of ILLITRACK runs, ballast pressure
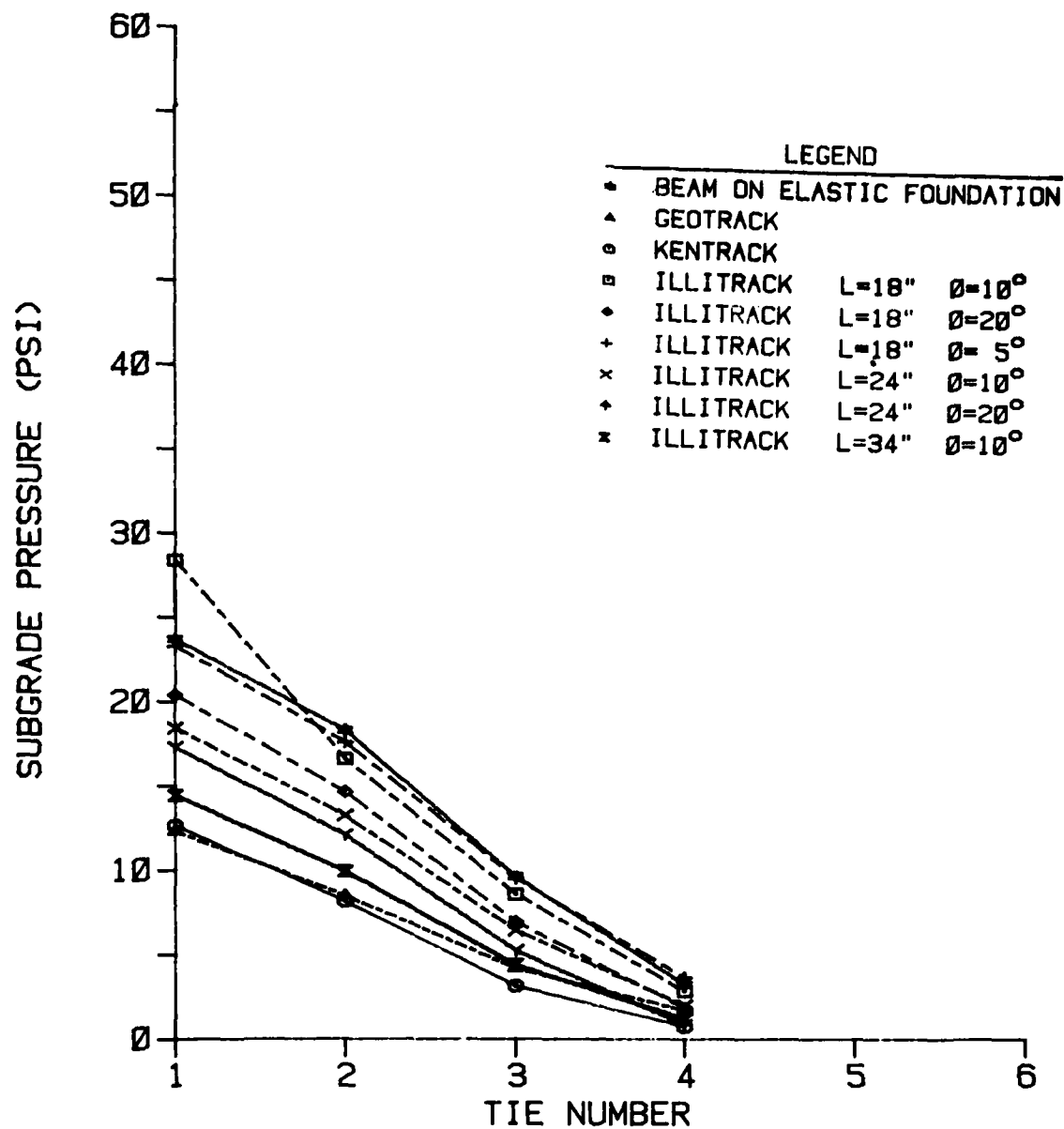
Figure 40.   Comparison of ILLITRACK runs, subgrade pressure

The differences in vertical pressure distribution are presented in Figure 41
again indicating the large variations that can be obtained by varying the L
and φ parameters.

VERTICAL STRESS (PSI)

LEGEND

▲	GEOTRACK		
⊙	KENTRACK		
▫	ILLITRACK	L=18"	Ø=10°
◆	ILLITRACK	L=18"	Ø=20°
+	ILLITRACK	L=18"	Ø= 5°
×	ILLITRACK	L=24"	Ø=10°
◆	ILLITRACK	L=24"	Ø=20°
✱	ILLITRACK	L=34"	Ø=10°
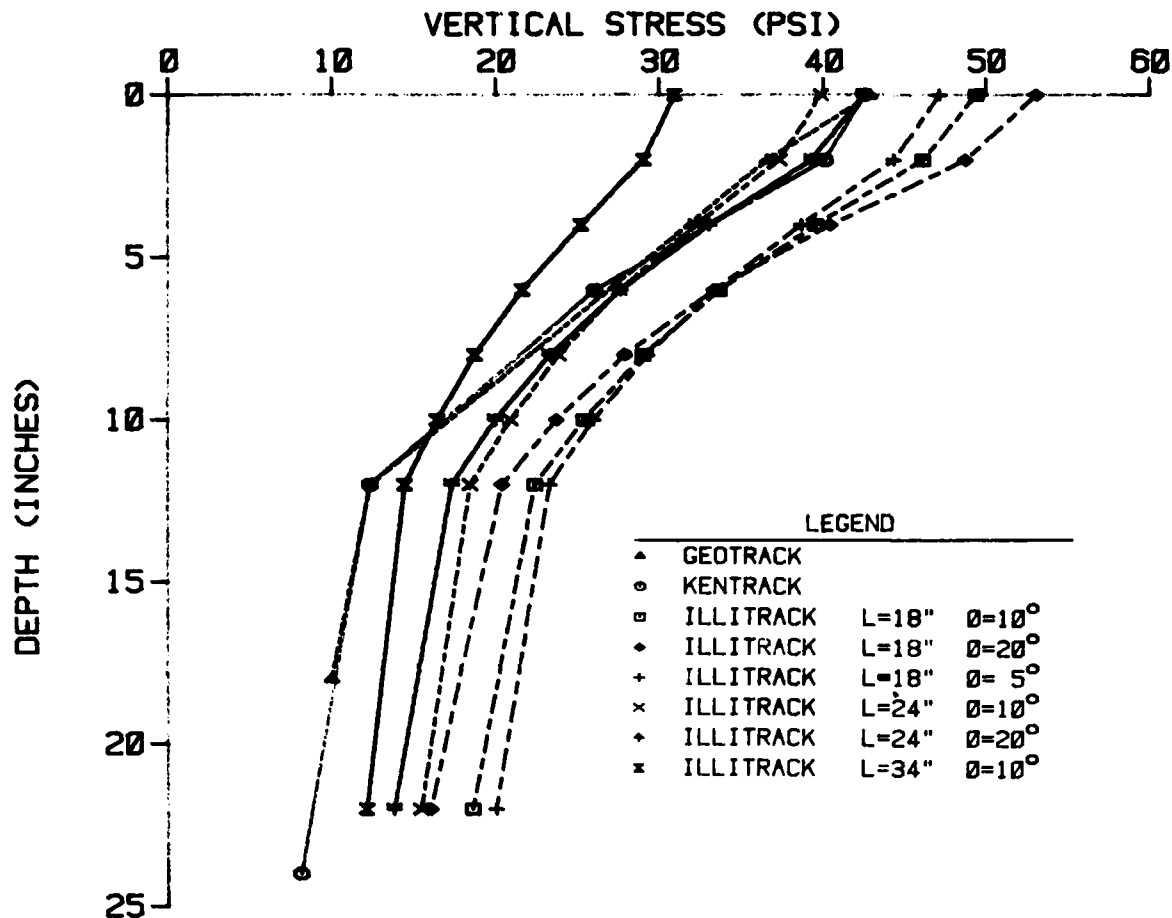
Figure 41. Comparison of ILLITRACK runs, vertical
pressure distribution

156. It is evident from the preceding discussion that the results ob-
tained with the ILLITRACK model can be varied greatly depending on the choice
of the L and φ parameters used in the analysis. The developers of ILLITRACK
have indicated that some variation of L and φ may be necessary in the
course of an analysis to obtain "reasonable" results. In addition, the L
and φ parameters for use in the model are essentially empirical and cannot
be determined from any conventional tests. The problems associated with
selecting and using the L and φ parameters in the ILLITRACK model are a
major drawback to the use of this model.

110

157. Program documentation. Up-to-date documentation of the computer program is essential to easy and efficient use of the model. Based on the test cases that were run in this study, an evaluation of the program documentation was made. The GEOTRACK documentation (Stewart 1981) is adequate for efficient use of the model, although it does not contain a complete example problem which would be helpful in using the model. The ILLITRACK documentation is found in "Finite Element Analysis of a Railway Track Support System" (Tayabji and Thompson 1976) with additional information published in a discussion by Raad and Thompson in "Study of Analytical Models for Track Support Systems" (Adegoke, Chang, and Selig 1979). This documentation is adequate for running the basic program; however, definite guidance on using the Mohr-Coulomb failure criteria is lacking. The KENTRACK program has good documentation in "KENTRACK, a Computer Program for Hot-Mix Asphalt and Conventional Ballast Railway Trackbeds" (Huang et al. 1984). This report details the development of the model and describes the system features, including input requirements, and printed output. Also described are the failure criteria and their use. Several example problems demonstrating the various available options are also given, along with a program listing. More information describing the actual uses of the various KENTRACK options as well as a better description of each of the input parameters would be helpful additions to this documentation.

158. Compatibility with evaluation methods. All of the candidate response models are adaptable to most of the structural evaluation methods that might be used in evaluating military trackage. The BOEF model is the most restrictive in that the inputs are limited to a load and the resulting deflection. While these inputs are relatively easy to obtain, the results of this analysis method are limited and do not provide a large amount of information on the ballast/subgrade response to the load. GEOTRACK, ILLITRACK, and KENTRACK all produce adequate information on the load response of the superstructure (rails and ties) and substructure. These computer models are compatible with any type of evaluation in that the results from a field investigation or laboratory testing or a combination of both may be used as input to the models.

Selection of Model

159. From the analysis of the candidate models and the discussions pre-
sented in the previous paragraphs the following conclusions may be drawn re-
garding the candidate track response models GEOTRACK, ILLITRACK, and KENTRACK:

a. The type of input data required for GEOTRACK, ILLITRACK, and
    KENTRACK is similar.

b. ILLITRACK requires a two-stage analysis to obtain complete
    results effectively doubling the amount of input required.

c. The maximum number of loads that may be evaluated with
    GEOTRACK is 4 compared with 25 for KENTRACK.

d. The allowable loading geometries limit the usefulness of
    GEOTRACK and ILLITRACK in evaluating multiaxle cases.

e. Both ILLITRACK and KENTRACK have failure criteria considera-
    tions incorporated into the models as optional features.

f. KENTRACK has the option of determining a damage analysis based
    on a failure criteria and the number of load repetitions using
    the structure during a specified period.

g. The time required for data preparation (including keypunching
    data cards) is approximately the same for GEOTRACK and KEN-
    TRACK with the preparation of a two-stage ILLITRACK run re-
    quiring approximately 50 percent longer.

h. A two-stage ILLITRACK run requires approximately 60 percent
    less CPU time than does the same GEOTRACK run. KENTRACK re-
    quires approximately 88 percent less CPU time than GEOTRACK
    and approximately 71 percent less CPU time than a two-stage
    ILLITRACK run. From this analysis KENTRACK is the most eco-
    nomical program to use.

i. The KENTRACK program outputs a larger variety of data than the
    other models; however, the ballast/subgrade response output is
    available only at the top and bottom of five user-specified
    layers.

j. The ILLITRACK program outputs a very large amount of data
    which the user must sort through to locate the desired values.

k. In GEOTRACK the ballast/subgrade response output is available
    for only five depths below each tie.

l. Analysis of the output from three different test cases indi-
    cates that track response predictions made with KENTRACK agree
    very well with predictions made with GEOTRACK and fairly well
    with predictions made using the BOEF analysis.

m. The ILLITRACK model predictions agreed fairly well with the
    BOEF analysis predictions.

n. The results of an ILLITRACK analysis can be varied greatly
    depending on the choice of the effective tie bearing length,

112

L , and angle of distribution, ϕ . The proper L and ϕ values to use are difficult to determine and the combination that results in the best determination of one parameter (rail moment for example) may not give the best determination of another parameter (such as subgrade pressure) for the same conditions. This is a major drawback to this model.

o. The program documentation for each of the candidate models is adequate. KENTRACK has the best documentation, followed by GEOTRACK, and then ILLITRACK.

p. All of the candidate models are compatible with current evaluation procedures.

160. Based on the criteria outlined in paragraph 143, experience using all three models, and the above conclusions drawn from analysis of the candidate models the KENTRACK model is judged to be the best model currently available for use in predicting the track response of Army trackage. The flexibility of this model, low cost to run, large variety of data output, and agreement with the results of both an established validated computer model (GEOTRACK) and the BOEF analysis are the primary reasons for selecting KENTRACK. Several minor modifications such as allowing the calculation of the ballast/subgrade response at several different depths instead of just at the top and bottom of specified layers would greatly increase the flexibility of the model.

161. The majority of track structure design and analysis worldwide is based on the BOEF theory, and therefore it will probably continue to be used as a standard. Because of this, the BOEF analysis should be retained as a means of determining track response, although KENTRACK has been selected as the primary track response model.

162. The comparison of track response models in this report is not a true validation of the KENTRACK model. Because of this, a full-scale field validation program should be conducted in which predicted track response would be compared with actual track response under a known load. This validation should be performed before the KENTRACK model is used extensively. If the validation indicates that the KENTRACK model does not best model the track response, the GEOTRACK model is recommended as an alternate model.

113

PART IX: SELECTION OF STRUCTURAL EVALUATION METHOD

163. Part VI of this report describes current methods for the structural evaluation of rail track structures. In the following paragraphs each of the evaluation methods will be evaluated and a selection made of one or more methods for additional field evaluation.

Criteria for Selection

164. The criteria for selection of a structural evaluation method for possible use in evaluating military railroads were:

    a. Time required for testing.

    b. Time required for track to be out of service.

    c. Cost of testing.

    d. Effectiveness in determining structural properties for use in a track response model.

    e. Amount of coverage obtained.

    f. Mobility of test system.

    g. Amount of track disturbance resulting from tests.

Selection of Structural Evaluation Methods

165. Based on the literature review performed in the course of this study, three methods of structural evaluation were chosen for further study. These methods are:

    a. Cone penetration testing.

    b. The ENSCO track stiffness measurement system.

    c. Load-deflection testing to determine material elastic properties.

The detailed field evaluation required to evaluate these methods is beyond the scope of this report and will be conducted in a separate study. Conventional test pits and laboratory tests such as described in paragraphs 97 through 101 should be used to determine the engineering properties of the track structure. These tests along with conventional track modulus tests and visual inspections will be used to determine the feasibility of using one of these test methods in a railroad evaluation procedure. Upon completion of the field evaluation of each of these test methods, one test method will be selected and an evaluation procedure will be developed around it.

114

PART X: TRACK EVALUATION FOR FUTURE MISSIONS

Overview

166. Current military preparedness missions require the use of the military railroad system to meet vital transportation needs in the event of war. The structural and functional ability of the existing trackage to meet these requirements is, for the most part, unknown. In addition, the heavier rolling stock now in the Army inventory (100- to 140-ton cars, for example) will lead to increased deterioration of the track structure with large amounts of traffic.

Effect of Heavy Axle Loads on Track Structure

167. In "Track Structures for Heavy Wheel Loads" Hay (1975) outlines the problems as well as some of the solutions associated with heavy wheel loads. Hay notes that a particular track structure has a load-bearing capability which is dependent on the combined characteristics of foundation, superstructure (rails and ties), and loads to be carried. Hay also indicates that the load-bearing capacity of much of the track currently carrying 90- to 100-ton cars has been exceeded.

168. Types of track deterioration that may be expected when inadequate track is subjected to heavy axle loads include: loss of line and surface, ballast and subgrade softening and pumping, wide gage, plate cut, split ties, spike-killed ties, rapid abrasive tie wear, battered rail ends, rail corrugation, and shelly formation on rails. Deferred maintenance of a track exposed to heavy wheel loads acts to compound the problem, as the heavy loads tend to intensify track deflection and differential movement between components, accelerating wear and deterioration.

169. To decrease the effect of heavy loads on the track structure, a stronger foundation is required. Hay recommends consideration of soil stability conditions and proper fill width and slope design, along with the application of soils engineering principles in the placing, compaction, and moisture content control during new construction to increase the foundation strength. Good drainage practices to prevent excess moisture from occurring in the subgrade are required. A filter blanket or a filter fabric is

sometimes required to prevent intrusion of the subgrade into the ballast. In existing track, Hay recommends taking advantage of improvements including:

  a. Adhering to good maintenance standards.

  b. Using a ballast depth consistent with the subgrade bearing capacity.

  c. Providing good drainage to keep the subgrade dry.

  d. Using rail weights consistent with expected loads and speeds in order to increase the load-bearing capability of the track structure.

Need for Evaluation

170. The existing conditions and load-carrying capacities of the majority of the military trackage are unknown. Before rational decisions can be made regarding track rehabilitation to meet future mission requirements, some type of structural evaluation must be performed. The evaluation should consider the physical properties of the materials comprising the track structure as well as the loads and traffic expected to use the track.

171. The immediate goal of railroad structural evaluation is to determine the adequacy of a particular track structure for supporting the given load and traffic conditions. A future goal of an evaluation program is the collection of data to provide the data base required to establish correlations between evaluation results, track conditions, and traffic in order to predict the performance of the track structure in terms of some specified future traffic conditions.

172. Selig, Yoo, and Panuccio (1982a) define track performance as "the degree of effectiveness with which a track system fulfills its intended purpose, that of providing the rail surface conditions necessary for the safe, confortable and economical operation of the trains." Indicators of in-service track performance are given as:

  a. Measurements of structural capability under load.

  b. Track physical appearance and its changes.

  c. The effects on track service such as safety, ride quality, and derailment frequency.

173. Selig, Yoo, and Panuccio list the most frequently used criteria for determining in-service performance as:

116

    a. Track stability as measured by the track modulus, track set-
        tlement, or track and tie resistance.

    b. Track geometry measurements.

    c. Safety as indicated by maximum allowable operating speed,
        speed restrictions, or derailment frequency.

    d. Ride quality as represented by frequency and amount of loading
        damage, or equipment deterioration.

    e. Maintenance requirements.

    f. Environmental effects such as vibration and noise transmitted
        to the surrounding community.

While all of these criteria are interrelated, track stability and track geom-
etry are the two major criteria used for determining performance.

Proposed Evaluation Program

174. As stated previously, the evaluation of military railroads should
be a combined structural and functional evaluation. Figure 42 presents a
flowchart showing the steps in the proposed railroad evaluation program. This
proposed evaluation program includes rail defect testing, geometry testing,
and visual inspection as well as structural evaluation to provide a complete
track evaluation. It is estimated that a complete evaluation as proposed here
would be performed approximately every 5 years. This evaluation program is
not intended as a substitute for the regular track inspections and maintenance
activities recommended in the "Army Rail Maintenance Standards" and normally
performed by an installation.

Structural evaluation

175. The structural evaluation phase of this evaluation program would
be accomplished using one or more of the test methods chosen for further study
and listed in paragraph 165. The details of the structural evaluation pro-
cedures will be developed in future research.

Functional evaluation

176. Measurements of geometric deviations made under load should be
included in the evaluation, along with visual inspections of turnouts and
crossings. While full-size track geometry cars are not practical for a large
part of the military trackage, high-rail geometry vehicles or possibly a
bolt-on package may provide adequate measurements of track geometry.

117
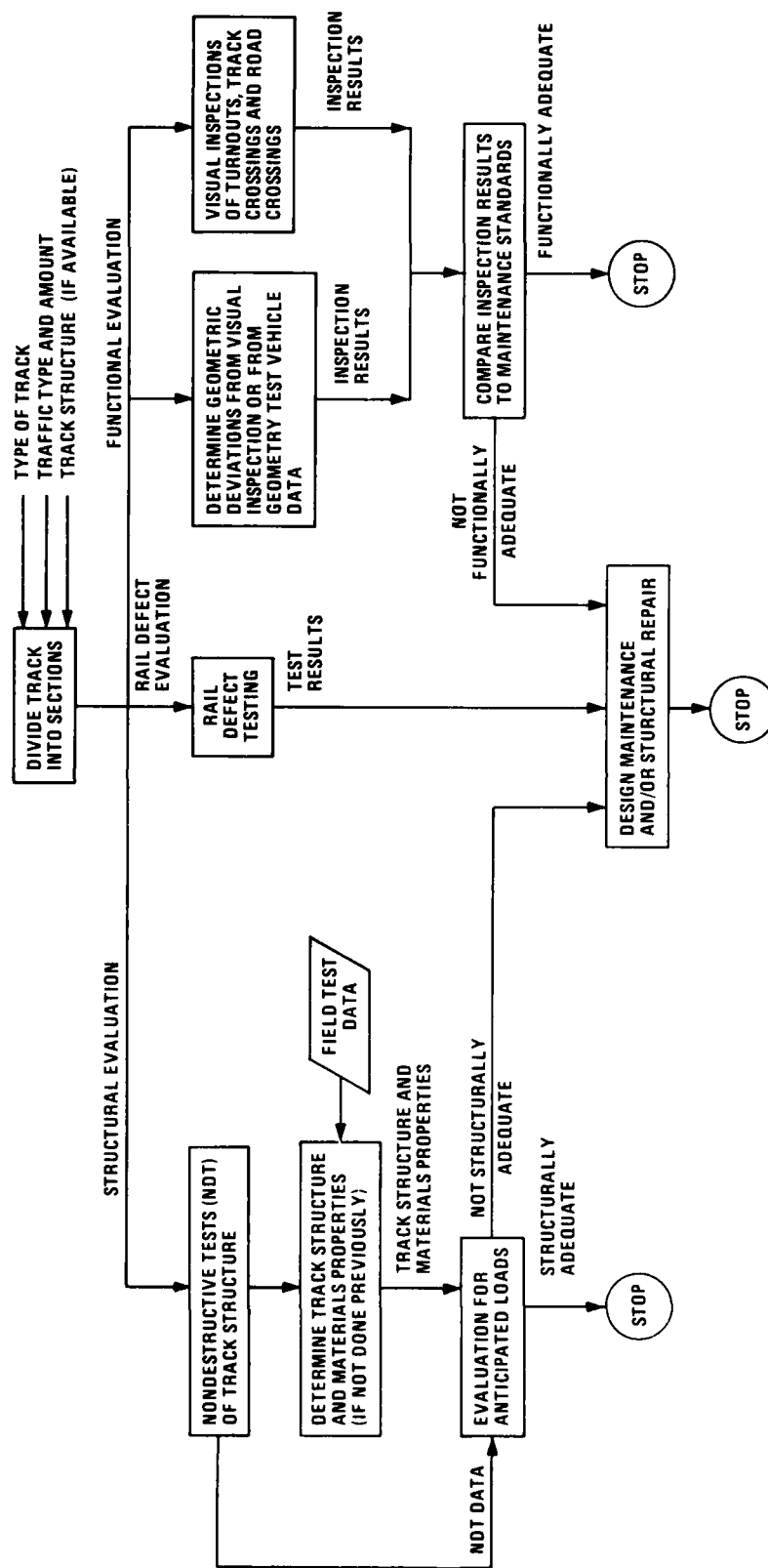
Figure 42. Proposed military railroad evaluation program

Uses for Evaluation Results

177. It is foreseen that the results obtained from an evaluation pro-
gram will be used as input for:

        a. Mission planning.

        b. Track rehabilitation.

        c. Track maintenance planning.

Data regarding the adequacy of a particular track and the need for upgrading
would be used in mission planning. Elements of the evaluation results used in
track rehabilitation include the structural requirements, the material re-
quired to meet the structural requirements, and based on these, the funding re-
quirements. Evaluation results can be used in track maintenance planning as
an input to a track management system, or can be used to estimate the struc-
tural, material, and funding requirements for adequate maintenance of the
track.

Requirements for Evaluation Program

178. Several tasks must be completed in order to implement the proposed
railroad evaluation program. These tasks are described below.

        a. Perform additional field validation of the KENTRACK track
            response model.

        b. Develop and validate the structural evaluation field pro-
            cedures and evaluation methodology.

        c. Investigate the feasibility of using a high-rail vehicle or
            bolt-on geometry package to measure loaded geometry.

        d. Adapt the currently available checklist for visual inspection
            of turnouts, rail crossings, and road crossings for use in
            the program.

        e. Use results from current lightweight rail life research being
            conducted at CERL to determine limits on the type and amount
            of internal rail defects allowed in Army trackage.

        f. Combine steps b through e above to yield the comprehensive
            evaluation procedure outlined in Figure 42.

PART XI: CONCLUSIONS, RECOMMENDATIONS, AND
AREAS FOR ADDITIONAL RESEARCH

Conclusions

179.   The major conclusions drawn from the study described in this
report are:

a. While a rational design procedure is available, most railroad
   track design in the United States is empirical, primarily
   based on experience with limited use of rational analytical
   methods.

b. Field or laboratory tests are seldom used to determine the
   physical properties or strength of subgrade materials prior
   to track design.

c. Most foreign design procedures determine track structure
   designs using rational analytic design methods combined with
   experience.

d. Most foreign design procedures emphasize the importance of
   classifying the subgrade soils and determining the subgrade
   strength.

e. The standard track sections required by foreign railroads are
   generally thicker than sections required by United States
   railroads which often have larger wheel loads.

f. Heavy wheel loads (100-ton cars for example) increase the
   amount and rate of deterioration that occurs in a rail track
   structure. Deferred or inadequate maintenance serves to com-
   pound this deterioration.

g. The deterioration due to heavy wheel loads can be limited by
   using good maintenance practices, using sufficient ballast
   depths for existing subgrade conditions, providing good drain-
   age, and using rail sizes consistent with expected loads and
   speeds.

h. The KENTRACK track response model is judged as the best model
   for predicting track response.

i. A complete field validation of KENTRACK should be conducted to
   ensure its adequacy at predicting field response.

j. Although models for predicting long-term track performance are
   being developed, only a rail performance prediction model is
   available at this time.

k. A railroad track evaluation program must include structural
   (load-carrying capacity) rail defect testing and functional
   evaluations in order to completely determine the capability of
   the track.

l. Cone penetration testing, the ENSCO track stiffness measurement system, and nondestructive load-deflection testing all have the potential for providing rapid, reasonably priced testing for the evaluation of track structures.

m. A full-size, track-bound track geometry car is not practical for use on the majority of military trackage.

n. A high-rail track geometry vehicle may be a better device for obtaining loaded geometry measurements than a full-size test car, although the load is small compared to actual wheel loads. A bolt-on geometry package that rapidly attaches to any type of rolling stock may be an even better method of determining loaded track geometry.

o. Rail defect testing, as has been performed on Army installations in the past, is an excellent means for locating internal rail defects that might cause a derailment.

p. A combined structural-functional evaluation program will provide vital input for use in mission planning, track rehabilitation, and track maintenance planning.

q. Lateral stability and/or rail rollover may be a problem on military trackage that carries heavy wheel loads, especially those tracks used by cars with three-axle trucks.

Recommendations

180. Based on the results of this study it is recommended that:

a. Track design procedures presented in "Manual For Railway Engineering" (American Railway Engineering Association 1982) be detailed in a document to aid the installations/districts in the design of military trackage.

b. The KENTRACK model be adopted as the primary model for predicting track response subject to favorable results of field validation. The BOEF analysis should be retained as a secondary response model.

c. A field evaluation of three evaluation methods (namely, cone penetration testing, the ENSCO track stiffness system, and nondestructive load-deflection testing) be made to determine the method best suited for the structural evaluation of track structures.

d. A structural evaluation method be developed based on the selected method.

e. An investigation be performed to determine the feasibility of using high-rail track geometry vehicles or a bolt-on type system to obtain geometry measurements under load on military trackage.

f. A comprehensive railroad evaluation program including both functional and structural evaluations be developed in order to allow a complete evaluation of military trackage. This includes completion of all the tasks outlined in paragraph 178.

121

Areas for Additional Research

181. During the course of this study several areas where additional re-
search is needed were observed. In direct relation to this study is the need
for development of a railroad structural evaluation procedure (recommendations
c and d above). This development will be performed as the second phase of
this project. In addition, a study to determine the feasibility of using
high-rail track geometry vehicles or a bolt-on system for making track
geometry measurements under load should be conducted (recommendation e) to
provide all the tools necessary to implement the proposed evaluation program.
Research is needed in the area of lateral stability and rail rollover to de-
termine if these areas are critical to the performance of military track.
Conversations with researchers at the AAR and with representatives of several
railroads indicated that some of the rolling stock currently in the military
inventory (especially the cars having three-axle trucks) may produce increased
lateral forces that subject the rail to overturning and result in a derail-
ment. Finally, a study should be performed to incorporate subgrade type and
strength into a rational design procedure similar to those used by many
foreign countries. A rational design procedure will provide better track
designs that should decrease maintenance requirements during the life of the
track.

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Table 1

AREA Specifications for Wooden Crossties*

Size	Sawed or Hewed Top, Bottom, and Sides			Sawed or Hewed Top and Bottom		
	Thickness in.		Top Width in.	Thickness in.		Top Width in.
1	6	×	6	6	×	6
2	6	×	7	6	×	7
3	6	×	8	6	×	8
3A		--		7	×	7
4	7	×	8	7	×	8
5	7	×	9	7	×	9

Table 2

AREA Recommended Gradations for Crushed Stone and Crushed Slag*

Size No.	Nominal Size Square Opening in.	Amounts Finer than Each Sieve (Square Opening) Percent by Weight									
		3 in.	2-1/2 in.	2 in.	1-1/2 in.	1 in.	/4 in.	1/2 in.	3/8 in.	No. 4	No. 8
24	1-1/2 to 3/4	100	90-100		25-60			0-10	0-5		
3	2 to 1		100	95-100	35-70	0-15			0-5		
4	1-1/2 to 3/4			100	90-100	25-55	0-15		0-5		
5	1 to 3/8				100	90-100	40-75	15-35	0-15	0-5	
57	1 to No. 4				100	95-100		25-60		0-10	0-5

* Used by permission from "Manual for Railway Engineering," Chapter 1 (American Railway Engineering Association 1984).

Table 3

AREA Recommended Gradations for Gravel*

Size No.	Percent Crushed Particles	Amounts Finer than Each Sieve (Square Opening) Percent by Weight							
		1-1/2 in.	1 in.	1/2 in.	No. 4	No. 8	No. 16	No. 50	No. 100
G-1	0-20	100	80-100	50-85	20-40	15-35	5-25	0-10	0-2
G-2	21-40	100	65-100	35-75	10-35	0-10	0-5		
G-3	41-75	100	60-95	25-50	0-15	0-5			

* Used by permission from "Manual for Railway Engineering," Chapter 1 (American Railway Engineering Association 1984).

Table 4

Recommended Allowable Tie Compressive Stresses, psi*

Wood Species	Continuously Dry	Wet Condition
Hardwoods		
White oak	550	365
Red oak	550	365
Beech	550	365
Birch	550	365
Rock elm	550	365
Hard maple	550	365
White ash	550	365
Pecan	660	440
Hickory	660	440
Black gum	330	220
Red gum	330	220
Tupelo gum	330	220
Yellow poplar	240	160
Softwoods		
Southern yellow pine	355	235
Longleaf yellow pine	415	275
Norway pine	240	160
Cypress	330	220
Hemlock	330	220
Douglas fir	310	205
Redwood	295	195
Eastern spruce	280	185

* Extracted from Table 1, "Manual for Railway Engineering" (American Railway
  Engineering Association 1984), Chapter 7-1-19, used by permission. Values
  given here are the unit stress in compression perpendicular to the wood
  grain. Where the original table gives more than one value, only the mini-
  mum value is presented here.

Table 5

Allowable Average Subgrade Bearing Pressures*

Subgrade Description	In-Place Consistency	Allowable Pressure Below Track psi	Source
Alluvial soils		Below 10	Clarke (1957b)
Made grounds, not compacted		11-15	
Soft clay, wet or loose sand		16-20	
Dry clay, firm sand, sandy clay		21-30	
Dry gravel soils		31-40	
Compacted soils		41 and over	
Well graded mixture of fine and coarse grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	65-100	US Navy (1971)
Gravel, gravel-sand mixtures, boulder-gravel mixtures (GW, GP, SW, SP)	Very compact Medium to compact Loose	55-85 41-55 25-50	
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact Medium to compact Loose	30-50 25-30 15-25	
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact Medium to compact Loose	25-40 15-30 8-15	
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very compact Medium to compact Loose	25-30 15-25 8-15	
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very stiff to hard Medium to stiff Soft	25-50 8-25 4-8	
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium to stiff Soft	15-30 8-25 4-8	
Coherent or fragmented rock		57	Milosevic (1969)
Banks of boulders		50	
Gravel		43	

(Continued)

* Adapted from DiPilato et al. (1983); used by permission.

Table 5 (Concluded)

Subgrade Description	In-Place Consistency	Allowable Pressure Below Track psi	Source
Dry clay and pug		28-36	
Fine sand		14-21	
Wet clay and pug		11-14	

Table 6

Japanese National Railways Standard Track Structure Sections

19,800-lb Static Wheel Load*

| Subgrade Type | Subgrade | | Ballast Thickness in. | Combined Subballast | | | |
	CBR	Liquid Limit		Slag Thickness in.	Crushed Stone Thickness in.	Sand Mat Thickness in.	Total Thickness** in.
Cut							
Sand I	2-4		10	6	10	0	26
Sand II	4-10		10	6	6	0	22
Sand III	10-20		10	6	0	0	16
Clay I	2-4	F60	10	6	10	6	32
Clay II	4-10	J60	10	6	6	6	28
Fill							
Sand	--		10	6	6	0	22
Clay	--		10	6	10	0	26

* Adapted from DiPilato et al. (1983); used by permission.
** Total thickness = ballast + slag + crushed stone + sand mat.

Table 7

Comparison of Track Response Models, Input Data

Input Parameter	Beam on Elastic Foundation	GEOTRACK	ILLITRACK*	KENTRACK
Problem title		X	X	X
Number of layers below tie		Maximum 5	Maximum 6	Maximum 6
Number of iterations for solution		Usually 3	Usually 3	NR
Consider tie-ballast separation?		Optional	NR	NR
Type of analysis: longitudinal or transverse		NR	X	NR
Total section depth		NR	X	NR
Layer material data (required for each layer)				
Layer thickness		X	X	X
Young's modulus		X	X	X
Poisson's ratio		X	X	X
Unit weight		X		NL
Coefficient of lateral earth pressure		X		NL
Coefficient for nonlinear (stress dependent) modulus calculation		X	X	NL
Data points for stress-dependent resilient modulus curves		NR	X	NR
Failure criteria			Optional	Optional
Angle of distribution, ϕ , for finite element calculations		NR	X	NR
Track modulus	X			
Tie data:				
Length		X	X	X
Width		X	X	X
Thickness			X	X
Spacing	X	X	X	X
Young's modulus		X	X	X
Poisson's ratio			X	
Moment of inertia		X	X	X
Unit weight		X		NL
Cross-sectional area		X		
Effective tie bearing length, L		NR	X	NR
Number of segments along tie		X		X
Tie spring constant			X	
Can nonstandard tie cross-sections be considered?		No	No	Yes
Location of rail on tie		X		X
Rail data:				
Weight		X		λ
Cross-sectional area		X		
Young's modulus	X	X	X	X
Moment of inertia	X	X	X	X
Section modulus				X
Gage		X		
Rail fastener stiffness		X		X
Loading conditions:				
Number of loads	Unlimited	Maximum 4	Not reported	Maximum 25
Load locations	X	X	X**	X
Load magnitude (wheel load)	X	X	X**	X
Load repetitions per specified period				Optional
Specified deflection (input instead of load)			Optional	
Calculation and output control:				
Tie numbers for data output		1-17	NR	2-11
Tie segment numbers for data output		X	NR	NR
Tolerance for stopping nonlinear analysis				X
Tolerance for stopping stress and strain calculations				X
Locations for horizontal tension calculation at layer boundaries				Optional
Locations for vertical compression calculation at layer boundaries				Optional
Depth for soil modulus calculation		X	NR	NR
Locations for soil stress calculations		X	NR	NR

Note: X = Input is required; NR = Input not required - program automatically considers; Optional = Input is optional depending on computations and output desired; NL = Input required only for nonlinear analysis.

  * Assuming both longitudinal and transverse analysis.

  ** ILLITRACK loading is considered symmetrical about the center line.

Table 8

Comparison of Track Response Models, Computer Time and Cost*

| Model | GEOTRACK | ILLITRACK | | | KENTRACK |
		Longitudinal	Transverse	Total	
Case 1 CPU time (sec)	88.092	24.660	12.096	36.756	10.656
Cost	$21.67	$6.07	$2.98	$9.04	$2.62
Case 2 CPU time (sec)	88.488	24.228	11.448	35.676	10.764
Cost	$21.77	$5.96	$2.82	$8.78	$2.65
Case 3 CPU time (sec)	89.424	24.192	11.484	35.676	10.620
Cost	$22.00	$5.95	$2.83	$8.78	$2.61
Case 4 CPU time (sec)	88.56	22.104	--**	--	16.452
Cost	$21.79	$5.44	--	--	$4.05

* Cost calculated assuming $0.246 per CPU sec.
** Transverse analysis not performed on Case 4.

Table 9
Comparison of Track Response Models, Output

Output	Beam on Elastic Foundation*	GEOTRACK	ILLITRACK	KENTRACK
ECHO Print of input		X	X	X
Rail moment:				
Maximum	X	X	S	S
At points along track	X		S	X
Rail deflection:				
Maximum	X	S	S	X
At points along track	X	X	S	X
Rail bending stress:				
Maximum	X			X
Along rail	X			X
Rail shear force	X			X
Rail seat load	X	X		
Tie deflection:				
At tie center line			S	S
At rail seat		X	S	S
Along tie			S	X
Tie bending moment:				
At tie center line		X	S	S
At rail seat		X	S	S
Along tie			S	X
Tie bending stress:				
Maximum				X
Along tie				X
Tie shear stress:				
Maximum				X
Along tie				X
Load on ballast from tie:				
Maximum		S	S	S
Along tie		X	S	X
Ballast pressure:				
Maximum	X	C	S	X
Along tie	X	C	S	X
Ballast deflection		S	S	S
Subgrade pressure	X	S	S	S
Subgrade deflection		S	S	S
Vertical displacement		X	S	X
Vertical stress		X	S	X
Horizontal stress		X	S	X
Vertical strain			S	X
Horizontal strain			S	X
Maximum principal stress		X	S	X
Intermediate principal stress		X		
Minimum principal stress		X	S	X
Shear stress			S	X
Shear strain			S	X
Bulk stress		X		
Direction cosines for principal stresses		X		
Geostatic vertical stress		X		
Equivalent triaxial stress states		X		
Track modulus		X		X
Young's modulus		X	S	X
Vertical grid lines			X	
Horizontal grid lines			X	
Element data			X	
Node coordinates			X	
Allowable repetitions				Optional
Damage analysis				Optional

Note: X = Direct output; S = Must be selected from output data; C = Must be calculated from output data.
   * All beam on elastic foundation output is hand calculated.

Table 10

Parameters Used in ILLITRACK Comparison

ILLITRACK Run	Effective Bearing Length, L	Angle of Distribution, ϕ
A	18	10
B	18	20
C	18	5
D	24	10
E	24	20
F	34	10

APPENDIX A: LITERATURE REVIEWED

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APPENDIX B: ORGANIZATIONS CONTACTED

American Railway Engineering Association
Washington, DC

Association of American Railroads Technical Center
Chicago, Ill.

Battelle Columbus Laboratory
Columbus, Ohio

Canadian National Railroad
Montreal, Quebec

US Department of Transportation
Transportation Safety Institute
Oklahoma City, Okla.

US Department of Transportation
Transportation Systems Center
Cambridge, Mass.

US Department of Transportation
Federal Railroad Administration
Office of Safety
Washington, DC

ENSCO, Inc.
Springfield, Va.

University of Illinois
Dr. M. R. Thompson
Urbana-Champaign, Ill.

Institute for Railroad Engineering
Dr. A. D. Kerr
Wilmington, Del.

Iowa Department of Transportation
Rail and Water Division
Ames, Iowa

Iowa State University
Dr. John Pitt
Ames, Iowa

University of Kentucky
Y. H. Huang, Dr. Jerry Rose, et al.
Lexington, Ky.

University of Massachusetts
Dr. E. T. Selig
Amherst, Mass.

Plasser American, Inc.
Chesapeake, Va.

Santa Fe Railroad
Chicago, Ill.

University of South Carolina
Dr. H. E. Stewart
Columbia, S. C.

Southern Railroad
Atlanta, Ga.

Sperry Rail Service
Danbury, Conn.

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